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NON-LINEAR DEFORMATION ANALYSIS OF AN EMBANKMENT DAM

BY

SILAS T. SANDERSON

A research project report submitted in partial fulfillment
of the requirement for the degree of

MASTER OF SCIENCE
IN
CIVIL AND ENVIRONMENTAL ENGINEERING

Project Advisor:
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Portland State University
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To my family, friends, and colleagues for their encouragement, advice, and timely distractions from the grind. And to Deanna Dsouza for having the confidence in me to complete this degree and for her intangible support throughout the years.

ABSTRACT

Non-linear deformation analysis (NDA) of embankment dams is becoming more common as geotechnical engineers seek to better understand how critical infrastructure will perform under high seismic loading. In order to perform this type of analysis, site specific ground motions and engineering properties of the embankment dam are required. The NDA modeling effort consists of a static and seepage analysis to establish an initial stress state followed by a dynamic analysis to simulate the embankment response to ground motions. Using this approach, a two dimensional NDA was performed on an embankment dam using the NDA program FLAC. This report discusses the input parameters, modeling efforts, and results of the analysis.

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1.0 INTRODUCTION

1.1 Purpose and Scope

The effects of strong ground motions from earthquakes on embankment dams has varied from little visible damage to failure. In the context of seismic performance of embankment dams, failure in this report is defined as breach of the embankment and loss of the reservoir either due to internal erosion or deformations at the crest that exceeds the available freeboard. Generally, embankment dams have performed well during seismic events (Foster et. al, 2000). The recent Tohoku earthquake (Mw 9, 2011) further affirmed this assertion. Over 400 dams experienced “severe” ground motions and only one irrigation dam failed (Matsumoto et. al, 2011). In the United States., a notable dam safety incident occurred at the Lower San Fernando Dam in 1971. Strong ground motions from a magnitude 6.5 earthquake caused the embankment to significantly deform. While the embankment did not breach, only a small remnant of it remained to retain the pool. Arguably, this incident served as a catalyst to the geotechnical engineering community to better understand and predict the effects of strong ground motions from earthquakes on embankment dams.

Many dams in the Pacific Northwest were built during an era that did not fully recognize the seismic hazard presented by the Cascadia Subduction Zone (CSZ) or crustal sources. Overtime, the understanding of the seismic hazards in the region has changed. Fortunately, the analysis methods and techniques used to better understand embankment behavior from seismic loading have evolved and improved. These include empirical correlations (Swaigood, 2003, 2014), pseudo-static analysis (Hynes-Griffin and Franklin, 1984), Newmark sliding block analysis (Newmark, 1965, Makdisi and Seed 1978, and Bray, Macedo, and Travarasrou, 2018), 2-D equivalent linear analyses (ELA) (QUAD4), and more advanced non-linear deformation analysis (NDA) methods using programs such as FLAC (Itasca, 2008). These more advanced methods are becoming more common as engineers seek to better understand how critical infrastructure will perform under high seismic loading. Programs such as Fast Lagrangian Analysis of Continua (FLAC) use finite difference techniques to numerically model complex soil behaviors experienced by an embankment during seismic loading. The embankment is modeled with elements and each element is assigned its own properties and a constitutive model to define its stress/strain behavior. While this method is more complicated, the biggest advantage is that an estimate of the deformation

magnitude and patterns of movement can be obtained, and a more realistic (non-linear) soil behavior can be modeled.

The purpose of this study was to conduct a non-linear deformation analysis of an embankment dam subject to seismic loading using the program FLAC and to document the steps required to perform a NDA so that others may use this report as a roadmap. The objective of the analysis is to estimate the maximum relative crest movements and to observe the patterns of deformation. Below are the general steps required to perform a NDA on an embankment dam and the section of the report where they are discussed.

Task	Objective
Section 2 – Development of Ground Motions	
Design Earthquakes	Determine design or analysis criteria for target ground motions (e.g. OBE, MCE, MDE).
Site Characterization and Seismic Setting	Describe seismic setting of the project site, examine foundation conditions and determine site class.
Seismic Hazard Evaluation	Conduct seismic hazard deaggregation to determine dominant seismic sources and characteristics.
Target Spectra Development	Develop design or analysis target spectra using a deterministic seismic hazard analysis (DSHA) or probabilistic seismic hazard analysis (PSHA) in accordance with the design or analysis criteria.
Ground Motion Selection and Scaling	Select representative ground motions. Scale or match ground motions to target spectra.
Section 3 – Static and Dynamic Embankment	
Analysis Cross Section	Select analysis cross section and determine embankment zones requiring soil parameters
Soil Properties	Select constitutive model(s) and determine required soil properties for each embankment zone including, but not exclusively, unit weight (saturated and dry), permeability, strength parameters, small strain shear modulus (Gmax), Poisson's ratio, G/Gmax damping curves.
Section 4 – Non-linear Deformation Analysis	
Grid Generation	Select appropriate element height and generate an efficient and representative grid for model based on the selected analysis cross section.
Initial Stress State	Establish initial stress state for dynamic loading by incrementally “building” the embankment and conducting a seepage analysis to establish pore pressure distribution.
Dynamic Analysis	Run selected scaled or matched time histories through model, tracking important parameters or response through time.
Results	Document and validate results of analysis, parametric studies, and constitutive model calibration.

2.0 GROUND MOTIONS DEVELOPMENT

2.1 Background

The project site for this study is located in the Western Cascade subprovince of the Cascade Range. The embankment at the project site is a 347-foot-high zoned, rockfill embankment structure with a gated spillway.

Design Earthquakes and Ground Motions

The design and evaluation of civil works structures consists of the Operating Basis Earthquake (OBE), the Maximum Design Earthquake (MDE), and the Maximum Credible Earthquake (MCE). The OBE is an earthquake that can be reasonably expected to occur within the service life of the project and is defined as having a 50% chance of occurrence in a 100 year return period. It is evaluated using a probabilistic seismic hazard assessment (PSHA). The performance requirement is that the project function with little or no damage and without interruption of function. The MCE is the largest earthquake that can be expected at the site and is evaluated using a deterministic seismic hazard analysis (DSHA) taken at the median plus one standard deviation (84th percentile) level. A PSHA informs the return period of the MCE. The MDE is the maximum earthquake for which the structure is evaluated. For critical, high hazard potential structures, the MDE is equivalent to the MCE and the performance objective is that the structure does not lead to catastrophic failure. Given the low (<0.1g) anticipated ground motions at the OBE, only the MCE will be analyzed.

2.2 Site Characterization and Seismic Setting

The dam is within the area influenced by the Cascadia Subduction Zone (CSZ) of the Pacific Northwest and the highly active area of Cape Mendocino, California. The Cascadia Subduction Zone is part of the worldwide system of active faults that subdivides the earth's crust into large moving plates. The Cape Mendocino area is a triple junction of three seismically active components (Cascadia Subduction zone, San Andreas Fault, and Mendocino Transform Fault). The CSZ is an 800 mile long fault extending from Cape Mendocino, California north to Vancouver Island, Canada. It defines the area where the offshore oceanic plate (Juan De Fuca Plate) is diving down (subducting) beneath the North American Continental Plate. Within the zone, three types of earthquake sources dominate the seismic

hazard at this site. The first source area is along the interface between the subducting oceanic plate and the overriding continental plate. In this area, mega-thrust earthquakes with a magnitude (M_w) between 8.5 and 9.2 are believed to have occurred about once every 450 to 550 years. The second source zone results from the downward moving ocean crustal slab bending and breaking up generating M_w 5 to about 7.5 earthquakes. These earthquakes are generally quite deep with focal depths of 25 miles or more. Deep intraplate earthquakes are smaller in size but occur more frequently than the mega-thrust events. The third source zone is the typical shallow crustal earthquakes magnitudes <1 to about 7.

Site Geology and Foundation:

Rock units in the area of the reservoir fall into two main groups, the Western Cascades and the High Cascades. Overburden in the dam site area consists of alluvium, slopewash, and pumice deposits. Most of the overburden, under the main embankment, was removed and the dam foundation consists of basalt flows and tuffs of the Little Butte Volcanic Series. All of the right abutment area was stripped to rock except for a gravel zone that is similar to the pit run gravel embankment and was left in place between Stations A24+50 to A26+75. Between Stations A13+50 to A14+90 and A15+75 to A17+00, a trench for the impervious core and minus 3-inch gravel zones were excavated approximately 15 feet into a soft to moderately hard tuff. Originally, the left embankment intracanyon basalt flow, or caprock, was to be left in place and used as the foundation. However, deficiencies in the caprock exposed during cleanup of the foundation necessitated its removal from the dam foundation. The caprock is underlaid by a buried terrace of fluvio-glacial gravels. This older alluvial interbed consists of poorly graded sandy gravel, cobbles and boulders with minor silt, and is 100 feet thick in a former buried channel near the present channel and averages about 60 feet in depth. A cutoff trench was excavated through the gravel unit, down to bedrock, so that the impermeable zone could extend down to bedrock through the gravel.

Site Classification:

There is no geophysical or in-situ data (SPT, V_s30 , etc) available to determine a site class. The main dam is founded on Basalt Flows and Tuffs (Figure 1 through Figure 3). A site class was inferred by comparing the geologic conditions of the site to a database of published reference sites. The database contains shallow seismic site characterization data, including V_s data and associated geologic conditions,

from 25 reference sites in Washington and Oregon. Where Tuff or Basalt was noted as likely underlying the reference sites, the Vs profiles were examined. At the depths where bedrock was likely encountered, Vs values ranged from 640 to 1600 m/s. Therefore, Site Class B/C ($V_{s30} = 760$ m/s) was selected for the site.

Seismic Hazard Evaluation:

An initial evaluation of the seismic hazard utilized the 2008 and 2014 U.S. Geologic Survey (USGS) PSHA data. The PSHA provides spectral acceleration estimates at a bedrock site (Site Class B/C; $V_{s30} = 760$ m/s) for varying return periods. At this site, the 2008 and 2014 PGA data are nearly identical and are plotted in Figure 4.

Deaggregation:

Graphical deaggregation of the hazard at the 144, 950, and 2475 Year Return periods are shown in Figure 6 through Figure 8 for the PGA. Table 3 illustrates the relative contributions, M_w , distance, and ϵ values of seismic sources for four return periods (144, 950, 2,475 and 9,950 Years). Cumulative Cascadia is the dominant source at all design criteria levels followed by random WUS Crustal Gridded. As the table indicates, 84th percentile CSZ ground motions have a return period between 2,500 and 10,000 years.

Source Characteristics:

Sources characteristics were developed from the deaggregation data. For the CSZ ground motions, a 9.0 magnitude (M_w) was assumed at a distance of 105 km. The PSHA data (USGS 2008) was used to estimate the return period of a median plus one standard deviation (84th percentile) CSZ earthquake and was determined to be slightly less frequent than a 2475 Return Period (Figure 4). Therefore, a rupture distance that was slightly closer than the 2475 Return Period deaggregation was selected. The focal or hypo-central depth was estimated to be 25 km based on the data presented in *Juan de Fuca slab geometry and its relation to Wadati-Benioff zone seismicity* (McCrory, 2012). The WUS Crustal Gridded source characteristics were assumed to be equal to the maximum M_w and minimum distance found in deaggregation data. This resulted in a 6.31 M_w at a distance of 13.2 km.

2.3 Target Spectra Development

For the purposes of this project, only the MCE target spectra was developed. The MCE was developed using a DSHA taken at the median plus one standard deviation (84th percentile) level. Target spectra were developed for the dominant seismic sources, CSZ and WUS Crustal Gridded. The MCE design spectra for a CSZ Interface Event was developed using four established Ground Motion Prediction Equations (GMPEs) and were weighted as shown in Table 1. These weights are the same as was used in the USGS2014 source model.

Table 1: CSZ DSHA Weighting Factors

GMPE	Weight
Atkinson and Boore, 2003	0.1
Zhao, 2006	0.3
BC Hydro, 2016	0.3
Atkinson and Macias, 2009	0.3

Target spectra for the less dominant, WUS Crustal gridded source was developed using the PEER Weighted Average of 2014 NGA West-2 GMPE spreadsheet. The GMPEs were weighted as shown in Table 2.

Table 2: WUS Crustal Gridded DSHA Weighting Factors

GMPE	Weight
ASK14	0.22
BSSA14	0.22
CB14	0.22
CY14	0.22
I14	0.12

The target spectra for each dominant seismic source are shown in Table 5.

2.4 Ground Motion Selection and Scaling

A total of 5 ground motions records, 4 for the CSZ and 1 for the WUS Crustal Gridded ground motions, were selected for matching and scaling. Selection criteria considered rupture distance, V_s of the site, and spectral shape relative the target spectrum. The ground motion records selected for the target spectra are shown in Table 4. The selected ground motions were then scaled to match the target spectra with a priority given to matching the PGA, and the periods from 0.5 to 1.5 seconds, which is the estimated range of natural period of the embankment. The target spectra, scaled ground motions, and geomean of scaled ground motions are shown in Figure 9 and Figure 10.

Table 3: Deaggregation of Seismic Hazard at Project Site (USGS 2008)

Design Criteria (Return Period)	Cumulative CSZ ₁			WUS Crustal Gridded			Was-Ore Cascades-W. and Ext. ₂			50-km Deep Intra Plate			Individual Fault Sources >2% ₃		
	PGA	0.5 sec	3.0 sec	PGA	0.5 sec	3.0 sec	PGA	0.5 sec	3.0 sec	PGA	0.5 sec	3.0 sec	PGA	0.5 sec	3.0 sec
OBE (144)	% 37.48 M=9.0; R(km)=120 ε -2.01	38.36 -1.95	43.23 -2.14	33.8 M=5.82-6.31; R(km)=52.1-75.8 -0.2	24.86 0.16	13.04 0.38	21.2 M=6.84; R(km)=81.6-108.4 -0.09	28.81 0.15	37.16 0.13	4.87 M=6.43-6.71; R(km)=157.5-162.3 0.66	4.62 0.5	3.4 0.71	7.82 M=6.76; R(km)=141.7 1.15	11.58 0.89	17.57 0.54
949	% 77.9 M=9.0; R(km)=112-116 ε 0.09	85.25 -1.95	93.46 -2.14	17.09 M=5.98-6.21; R(km)=18.6-19.1 0.32	9.16 0.54	<3% - -	3.87 M=6.97-7.08; R(km)=43.0-55.2 1.4	4.25 1.45	3.11 1.58	-			-		
2,475	% 82.68 M=9.0; R(km)=106-137 ε 0.69	90.8 0.78	97.76 0.61	15.6 M=6.06-6.31; R(km)=13.2-13.6 0.58	7.44 0.78	<3% -	-			-			-		
9,950	% 82.64 M=9.0; R(km)=103.7-109 ε 1.3	92.69 1.5	97.32 1.33	14.9 M=6.16-6.41; R(km)=9.3-9.8 0.96	7.12 1.1	<3% -	-			-			-		

1. Cascadia Megathrust and Floating M8.0-M8.7. M, R, and ε values are for the dominant Cascadia Megathrust source
2. Wash-Oreg Cascades-West and Extensional Faults. M, R, and ε values are for the dominant Cascades-West source
3. Klamath Graben, Sky Lakem Whaleshead, and Cape Blanco Anticline. M, R, and ε values are for the dominant Whaleshead source
4. MCE Design Level estimated return period between 2500 and 10000 Years.

Table 4: Source Characteristics, Summary of Selected Ground Motion Records, and Scaling Factors

Source to Match	Earthquake	Year	Mw	Station	Component	Fault Mechanism	Epi. Distance, km	Vs30, m/s	Scale Factor
CSZ	Tohoku	2011	9	TAJIRI-MYGH06	90	Subduction - Interface	117.2	593	1.1
M	Tohoku	2011	9	SHIZUGAWA-MYGH12	360	Subduction - Interface	88.3	748	0.75
R, km	Maule	2010	8.8	HUALANE-HUA	T	Subduction - Interface	136	528	0.7
105	Maule	2010	8.8	CURICO-CUR	NS	Subduction - Interface	170.5	592-710	0.9
WUS Crustal Gridded	Iwate - Japan	2008	6.9	AKT023	EW	Reverse	11.68	556	0.78
M					NS				0.78

Table 5: Deterministic Hazard Spectra, Site Class C/D (5% Damping)

T (s)	CSZ (med + σ) Sa (g)	Gridded (med + σ) Sa (g)
0.01	0.329	0.299
0.02	0.346	0.306
0.03	0.362	0.341
0.05	0.384	0.439
0.075	0.492	0.574
0.1	0.592	0.660
0.15	0.666	0.737
0.2	0.668	0.710
0.25	0.642	0.639
0.3	0.620	0.572
0.4	0.589	0.465
0.5	0.538	0.385
0.75	0.449	0.255
1	0.391	0.183
1.5	0.305	0.106
2	0.245	0.071
3	0.173	0.039
4	0.130	0.025
5	0.104	0.017
7.5	0.069	0.008
10	0.052	0.005



Figure 1: Embankment Foundation Geology, Sta. 10+50 to 21+00

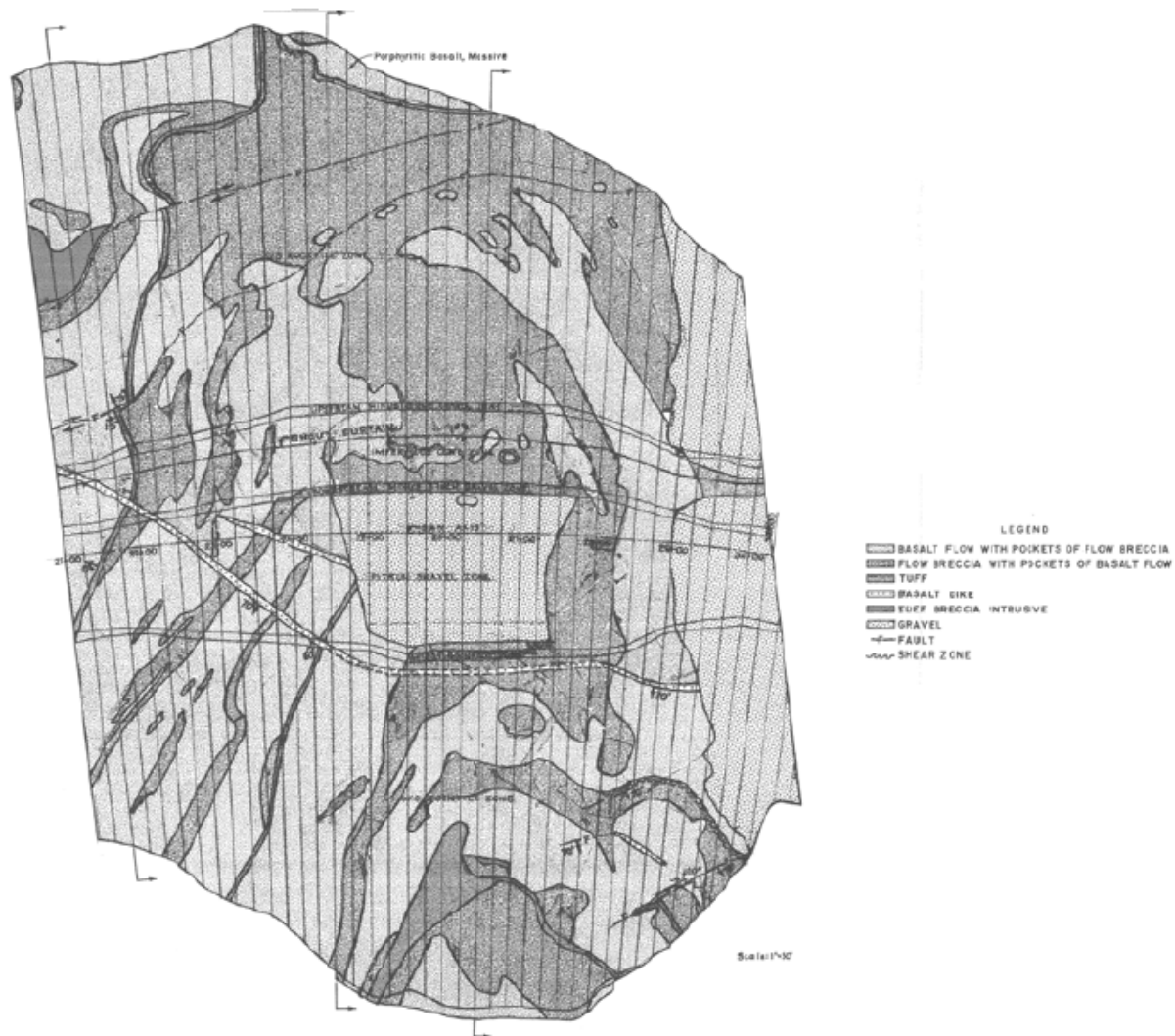


Figure 2: Embankment Foundation Geology, Sta. 21+00 to 30+30

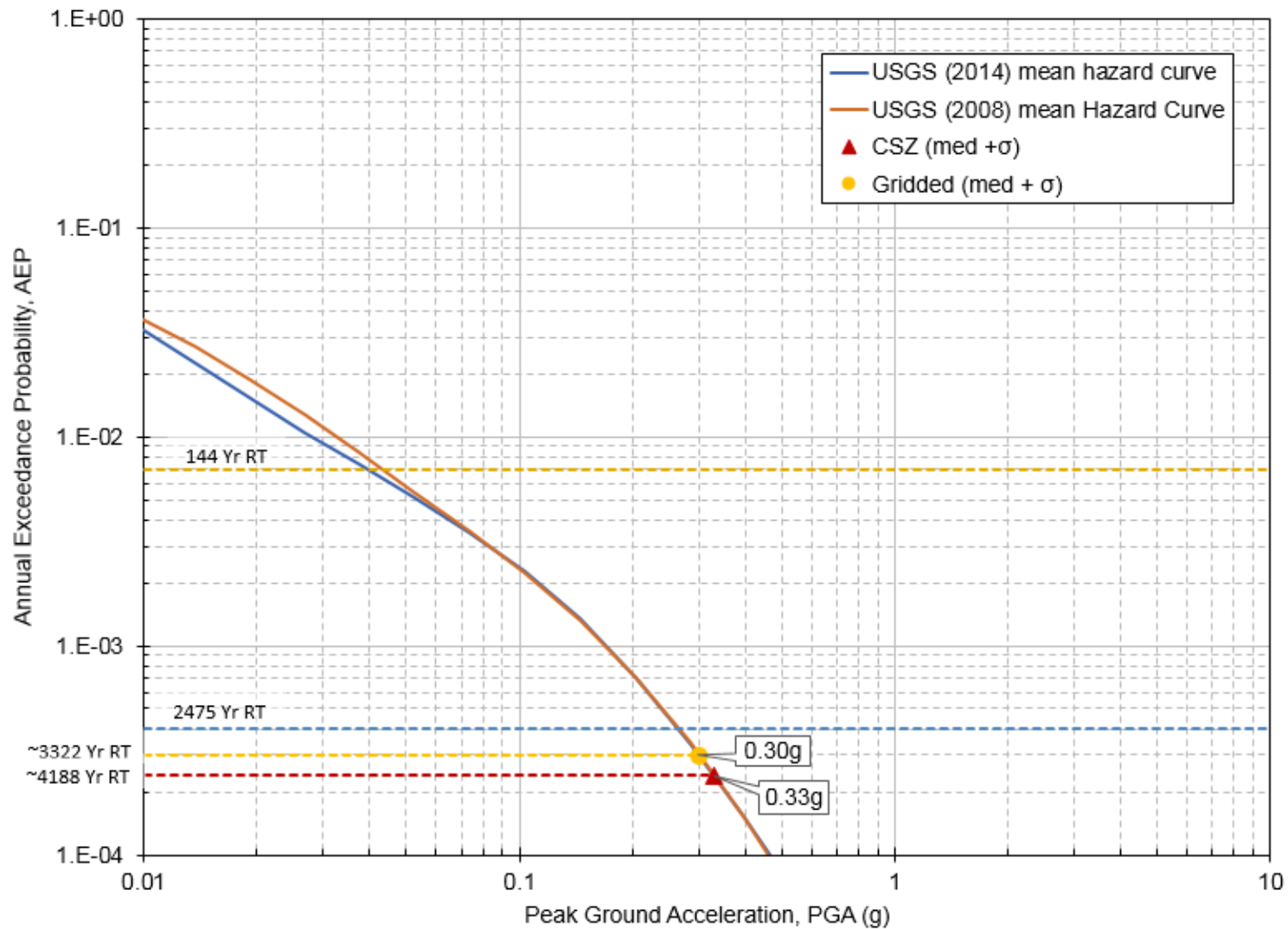


Figure 4: Mean Hazard Curves for PGA (USGS 2008 and 2014), and CSZ (median + σ) and WUS Crustal Gridded (median + σ) Deterministic PGA for Site Class B/C ($V_s=760$ m/s), 5% Damping.

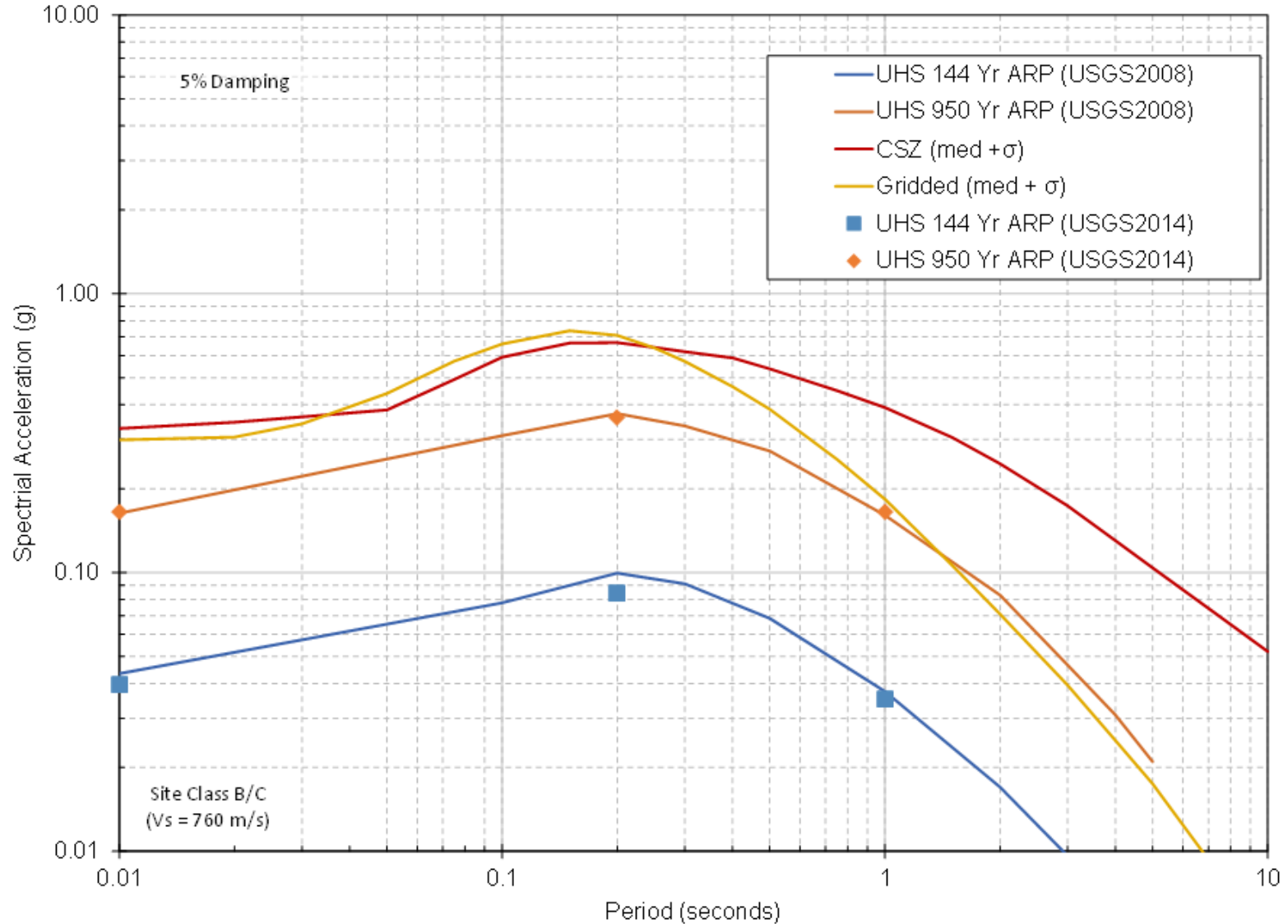


Figure 5: CSZ (median + σ) and WUS Crustal Gridded (median + σ) Deterministic Target Spectra for Site Class B/C ($V_s=760$ m/s), 5% Damping (50% in 100 Years and 10% in 100 Years Uniform Hazard Spectra (USGS 2008 and 2014) provided for comparison).

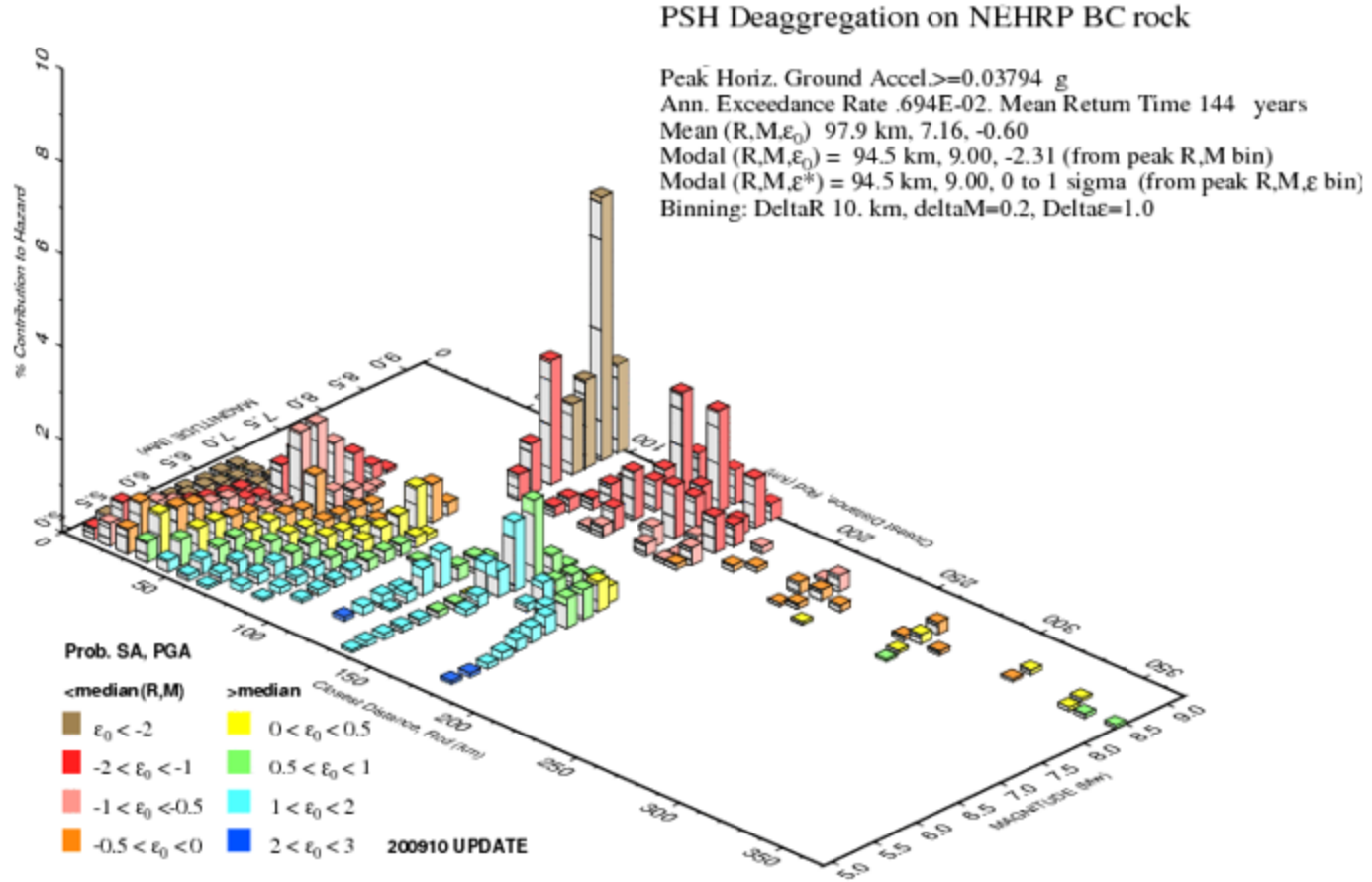
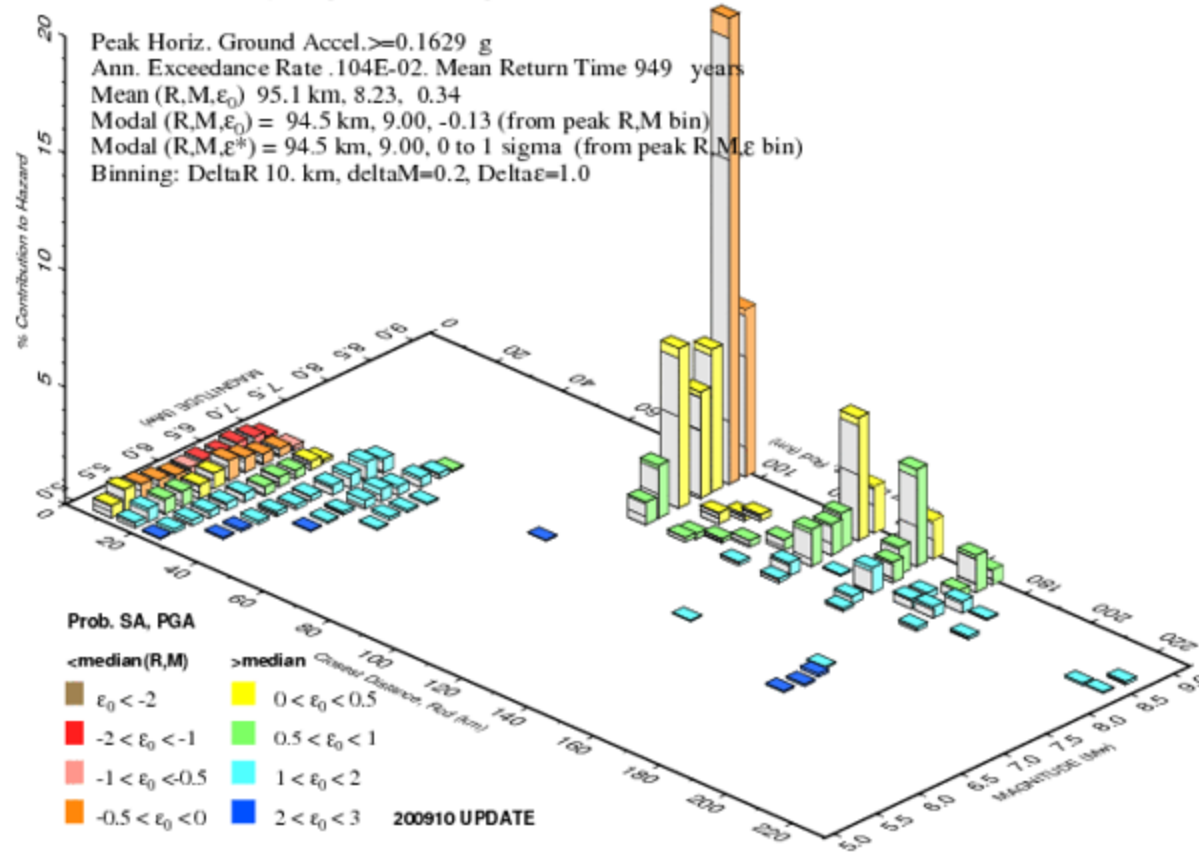


Figure 6: Deaggregation of Hazard by Magnitude and Distance at PGA 50% in 100 Years (144-Year Return Period).

PSH Deaggregation on NEHRP BC rock



GMT 2017 Mar 13 16:52:29 Distance (R), magnitude (M), epsilon (E), deaggregation for a site on rock with average vs=350. m/s top 30m. USGS CSH-IT PSH-2008 UPDATE Bins with 110.09% contrib. omitted

Figure 7: Deaggregation of Hazard by Magnitude and Distance at PGA 10% in 100 Years (949-Year Return Period).

PSH Deaggregation on NEHRP BC rock

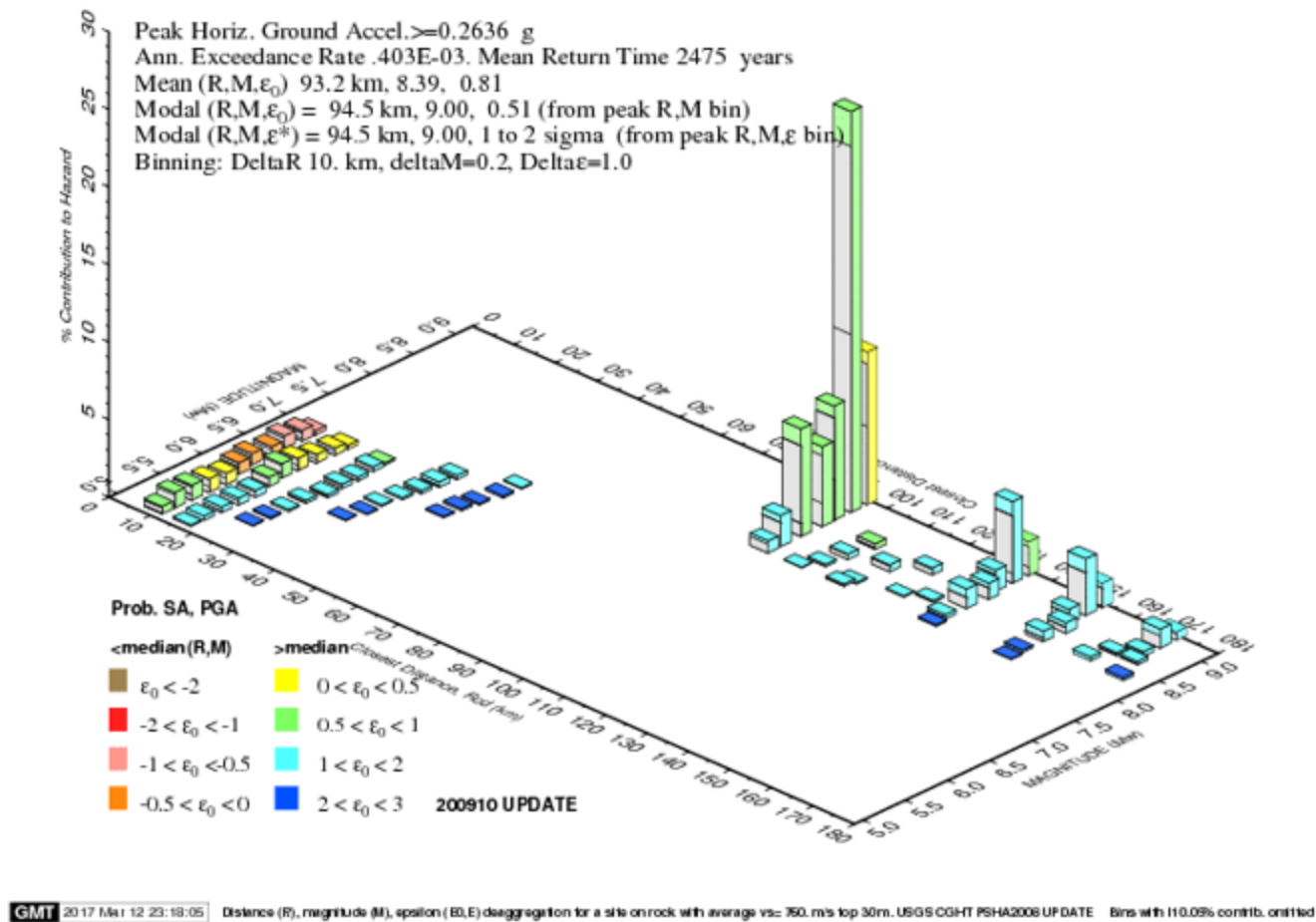


Figure 8: Deaggregation of Hazard by Magnitude and Distance at PGA 2% in 50 Years (2475-Year Return Period).

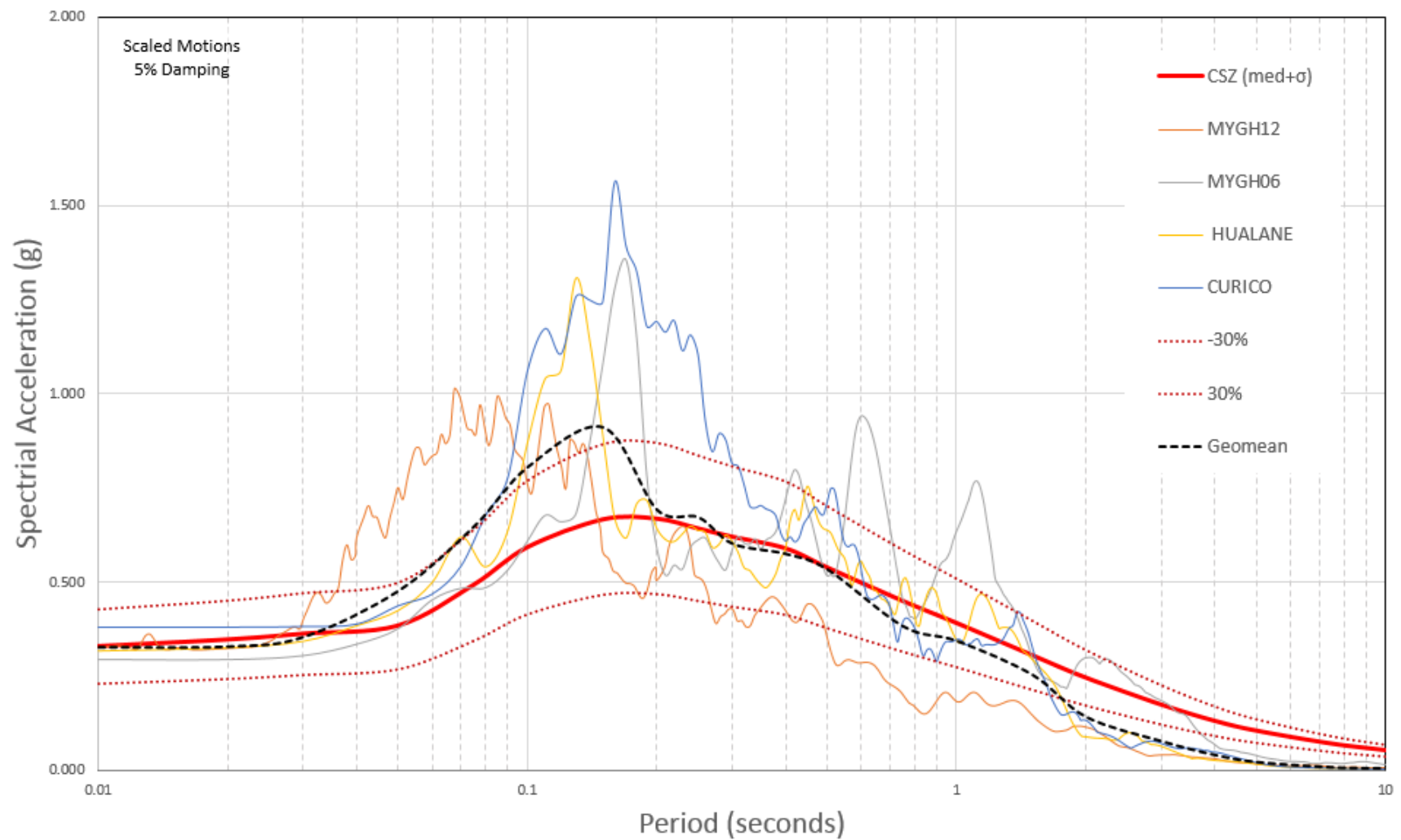


Figure 9: Comparison of Selected Ground Motions Records (Scaled) and geomean with the Deterministic Target CSZ (median + σ) Spectrum, 5% Damping.

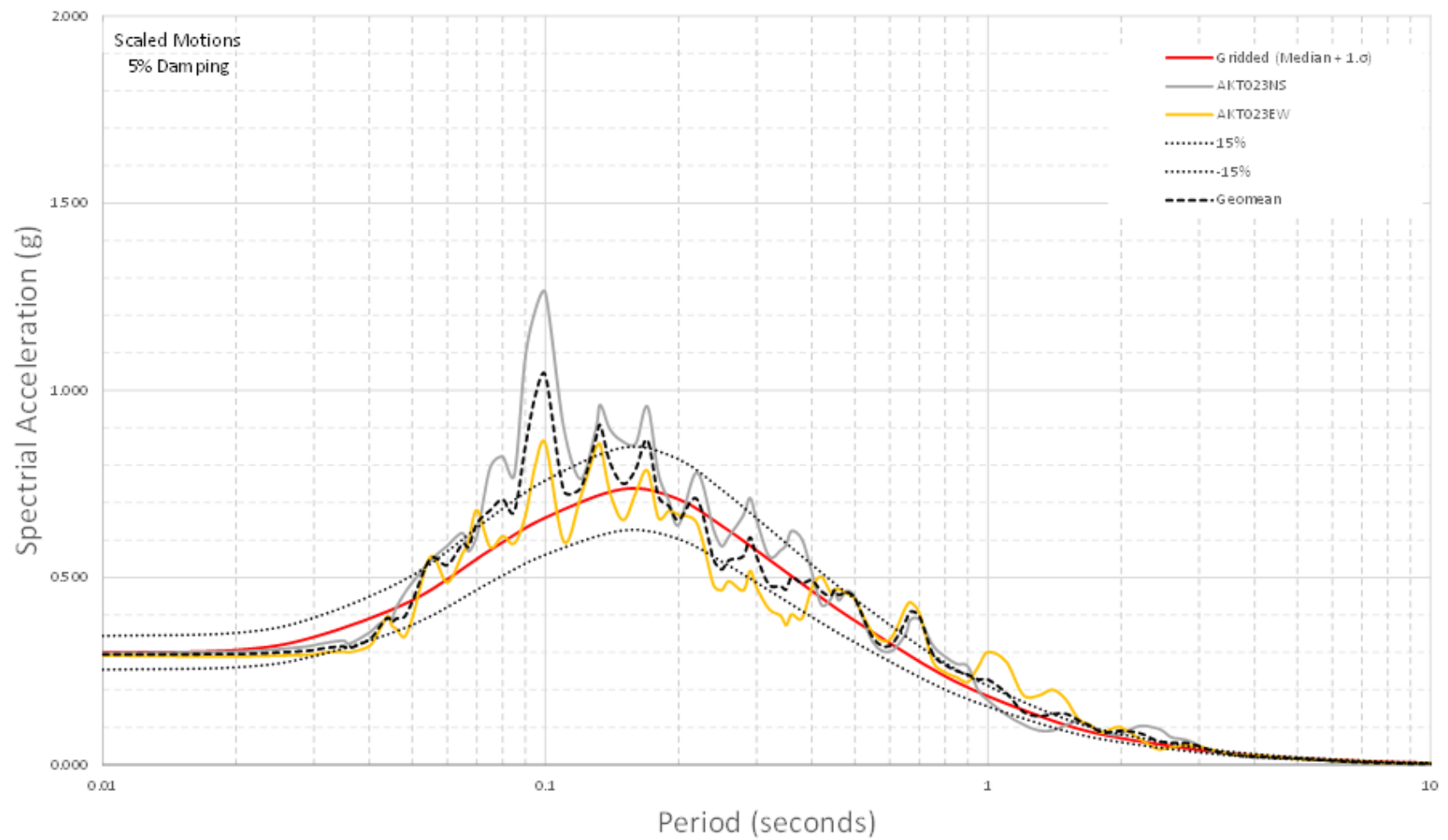


Figure 10: Comparison of Selected Ground Motions Record (Scaled) with the Deterministic Target WUS Crustal Gridded (median + $σ$) Spectrum, 5% Damping

3.0 STATIC AND DYNAMIC EMBANKMENT PARAMETERS

3.1 Cross Section

The embankment dam analyzed is approximately 347 feet tall and 3600 feet long. The maximum section is located at Sta. 26+00 and was used in the analysis. The bedrock dip at the project site is relatively minor, therefore a flat bedrock profile was assumed. The embankment cross section generally consists of a slightly inclined, central impervious core with upstream and downstream rockfill shells. The impervious core is protected by upstream and downstream 10 foot filter zones. A sizeable pit run gravel section exists just downstream of the impervious core and filter. A 10 foot wide select rockfill zone transitions the pit run zone to the outer rockfill shell. The upstream slope is inclined at 1.85H:1V while the downstream slope is inclined at 1.75H:1V.

3.2 Soil Properties

Key embankment materials required for the deformation analysis include unit weights, permeability, and static strength parameters. The assigned properties for each embankment zone are discussed below.

Unit Weights

The impervious core material was sourced from necessary excavations, left abutment and terrace excavations, and approved upstream borrow sources. The core material can generally be classified as Clay (CL) with some silt (ML) and minor amounts of plastic silt (MH) and clay (CH). The Plasticity Index ranges from 10 to 32 with an average of 22. Construction specifications required the core material to be compacted in 1 foot layers with a maximum particle size of 6 inches. Compaction was achieved with a minimum of four passes of a rubber tire roller. In place field density and moisture data indicates that the embankment was compacted to between 75 and 106 pounds per cubic foot (pcf) with an average of 95 pcf. Field dry density ranged from 88 to 107 percent of Standard Proctor optimum dry density. Moisture content ranged from 19.7 to 37 percent with an average of 27 percent which corresponds to no more than

3 percent above optimum and no less than 1 percent below optimum. The average field saturated unit weight was 123 pcf.

Upstream and downstream filter zones consist of 3-inch minus materials processed from gravel and concrete borrow. The maximum particle size is 3 inches with no more than 5% passing a No. 200 sieve. Compaction was achieved in 1 foot lifts with a minimum of 2 passes of a vibratory roller. The average field dry density was 132 pcf with a range of 112 to 140 pcf. The average saturated unit weight was 147 pcf and the average moist unit weight was 130 pcf.

The pit run gravel was sourced from foundation excavation and designated upstream borrow sources. The maximum particle size was 15 inches with no more than 5% passing the No. 200 sieve and no more than 60% passing the No. 4. Compaction was achieved in 1.5 foot lifts with no less than 2 passes of a vibratory roller. The average field density was recorded as 142.5 pcf with a range of 136 to 149 pcf. This is was significantly higher than the design density of 117 pcf. The average moist and saturated units weights were recorded as 147 pcf and 153 pcf, respectively.

Rockfill and select rockfill was obtained from sound fragments of basalt from quarry operations or required excavations. Oversize cobbles from 3-inch minus processing were also used. The maximum rock size was not to exceed 2 feet, the average D50 was 3.5 inches, and no more than 12% was allowed to pass the No. 4 sieve. Compaction was achieved in 2 foot lifts for the rockfill and 1 foot lifts for the select rockfill. Both the rockfill and select rockfill were compacted with no less than 4 passes of a vibratory roller. The field dry density was recorded to be 127 pcf which was greater than the design dry density of 116 pcf. The average moist and saturated units weights for both the rockfill and select rockfill were recorded as 127 pcf and 141 pcf, respectively.

The main dam is founded on Basalt Flows and Tuffs. According to project design records and sampling, the average dry density and saturated density are 127 pcf and 141 pcf, respectively.

Permeability

Design documentation used a permeability of 1.97×10^{-8} ft/min (1×10^{-8} cm/s) for the impervious core. This value is reasonable and will be used in the analysis. There is no design documentation for the permeability of filter zones. The filter zones were specified to have less than 5% passing the No. 200. This material can be described as a well graded mixture of sand and gravel which given the limited fines

content should be free draining. Therefore, a permeability of 1×10^{-3} cm/s was selected. There is no design permeability data for the pit run gravel. The sand content of the pit run gravel ranges from 0 to 45% and there are limited (<5% fines). Therefore it is reasonable to assume that this zone would be an order of magnitude more pervious than the filter zone and will be assumed to be 1×10^{-2} cm/s. The rockfill and select rockfill consists of mostly cobbles and gravels with 0 to 12% sand. This can be considered to be a fairly clean, gravel with high permeability. Therefore it is reasonable to assume a permeability of 1×10^{-1} cm/s. Based on design documentation, the permeability of bedrock is 1.7×10^{-4} ft/min (8.64×10^{-5} cm/s).

Static Strength Parameters

The project design and field strength data was available for the impervious core material. Only design strength data was available for all other zones. Design and field strength data are presented in Table 6. For the purposes of this study the Design values (R strength) were used for the impervious core since they are fairly similar to the field data and they are conservative when compared to the S strength data. The remaining embankment materials were reviewed and the S strength (Consolidated-Drained) values were used for the analysis.

Table 6: Static Strength Soil Parameters – Design and Field Records

Material		Test	Φ (°)	c (ksf)
Impervious Core	Design	R	13.5	0.6
		S	25	0
	Field Data	R	9.9 – 15.8	1.08
		S	23.6 – 34.2	0
Minus 3 Inch Filter	Design	R	39	0
		S	42.5	0
Pitrun Gravel	Design	R	39	0
		S	42.5	0
Rockfill and Select Rockfill	Design		45	0

The soil parameters used in the analysis are presented in Table 7.

3.3 Dynamic Properties

In order to perform the analysis, several dynamic soil properties are required. The dynamic properties required include maximum shear modulus (G_{max}) and the G/G_{max} and damping ratio curves. Site specific data was not available. However, a nearby project with similar materials and in place densities has site specific shear wave velocity data available that was used to inform the values, where appropriate, listed below. The constitutive model for all embankment materials is a Mohr-Coulomb with sigmoidal-3 function used to approximate the G/G_{max} curves.

G_{max}

G_{max} values were determined using the following relationships.

Impervious Core: As stated above, the average PI of the core material is 22. Given the relatively short height of the embankment and the compaction equipment used, the over consolidation ratio (OCR) is likely between 2 and 3. Therefore the following relationship will be used:

$$G_{max}/S_u = 600 - \text{Kramer (1996)}$$

Pit Run Gravel, Rockfill, and Filters: $G_{max} = 1000(\sigma'_m)^{0.5}k_{2,max} - k_{2,max} = 120$. In order select an appropriate $k_{2,max}$ value, a comparison was made between a $k_{2,max}$ values of 80 and 120. Shear wave velocities were then calculated and compared to the nearby reference site with similar embankment construction. The $k_{2,max}$ value of 120 was chosen because it most closely compared to the site specific shear wave data collected at the reference site.

G/G_{max} and Damping Curves

Impervious Core: Curves by Vucetic and Dobry (1991) for PI = 15. Avg. PI = 22 (10-32 range)

Filters and Pit Run Gravels: Curves by EPRI (1993) for depths = 50-120 feet.

Rockfill: Gazetas and Dakoulas (1992).

Table 7: Analysis Soil Properties

Soil Properties								
Unit	Description	Gmax (psf)	γ_{Moist} (pcf)	γ_{Sat} (pcf)	c or Su (ksf)	ϕ	G/Gmax	Vertical Perm. K (cm/s)
Impervious Core	CL and ML; $PI_{\text{Avg}}=22$; Compaction: 1 foot lifts; 4 passes rubber tire roller; +3/-1% opt.	Su(600) - Kramer (1996)	121	123	0.6	13.5	Vucetic and Dobry (1991)	1x10-8
Minus 3 Inch Filter	U/S and D/S filters; Max. size 3 inches; Compaction: 1 foot lifts; 2 passes vibratory roller	$1000(\sigma'_{\text{m}})^{0.5} k_{2,\text{max}} - k_{2,\text{max}} = 120$	130	147	-	42.5	EPRI (1993) - 50-120 ft	1x10-3
Pitrun Gravel	Max. size 15 inches; Compaction: 1.5 foot lifts; 2 passes vibratory roller	$1000(\sigma'_{\text{m}})^{0.5} k_{2,\text{max}} - k_{2,\text{max}} = 120$	147	153	-	42.5	EPRI (1993) - 50-120 ft	1x10-2
Rockfill and Select Rockfill	Max. size 24 inches; d50=3.5 inches; Compaction: 2 foot lifts (1 ft select); 4 passes vibratory roller	$1000(\sigma'_{\text{m}})^{0.5} k_{2,\text{max}} - k_{2,\text{max}} = 120$	127	141	-	45	Gazetas and Dakoulas (1992)	1x10-1
Bedrock	Basalt Flows and Tuffs	-	127	141	-	-		8.64x10-5

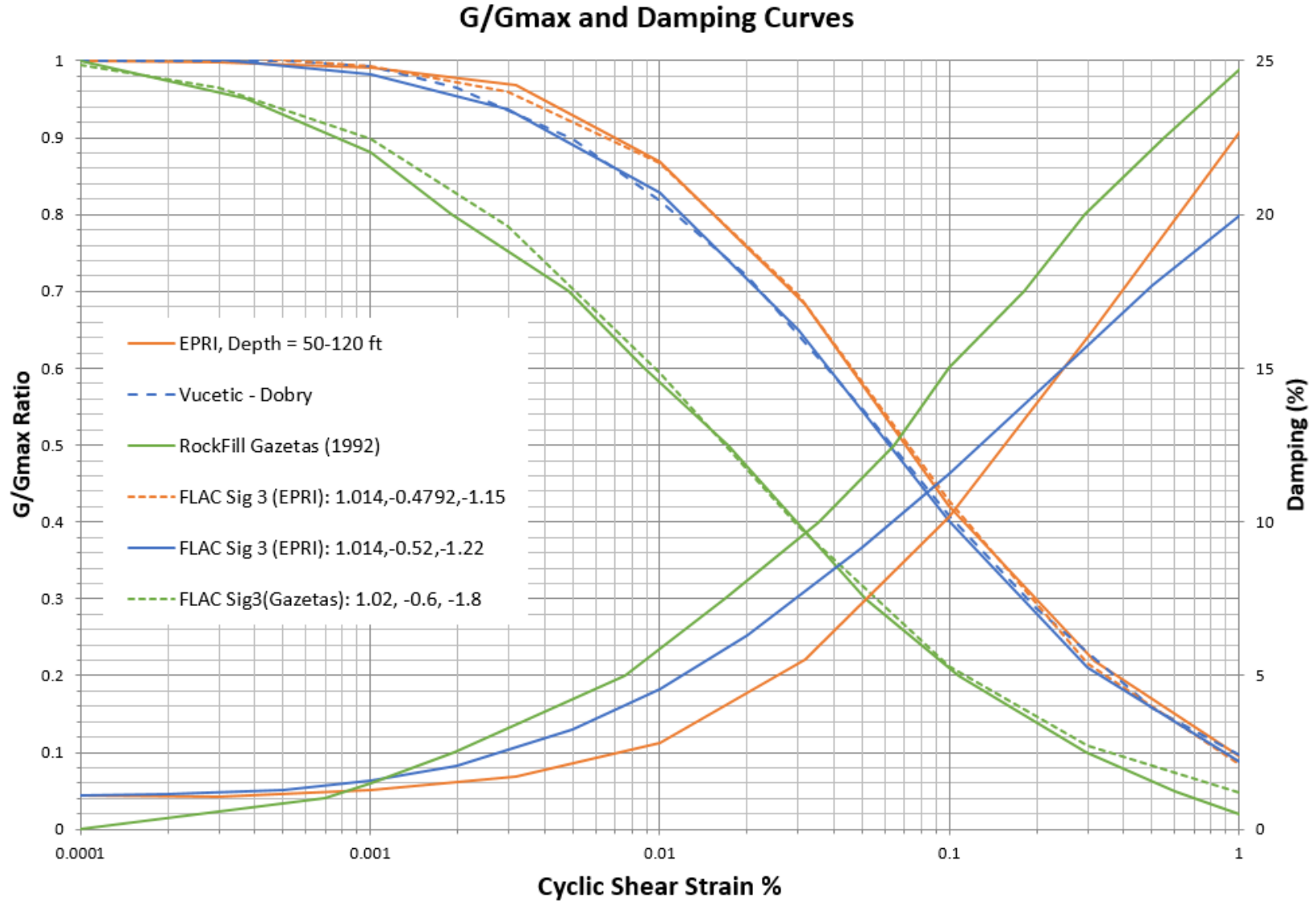


Figure 11: G/Gmax and Damping Curves

Table 8: Filter, Rockfill, and Gravel k_{2,max} comparison

Filter, Rockfill, and Gravel					
σ' (ksf)	σ'm (ksf)	k _{2,max}		k _{2,max}	
		80	120	80	120
		G _{max} (psf)		V _s (ft/s)	
0.0	0.0	0.0E+00	0.0E+00	0	0
0.4	0.2	1.2E+06	1.8E+06	510	624
0.8	0.4	1.7E+06	2.5E+06	606	742
1.2	0.7	2.1E+06	3.1E+06	671	822
1.6	0.9	2.4E+06	3.6E+06	721	883
2.0	1.1	2.7E+06	4.0E+06	762	934
2.4	1.3	2.9E+06	4.4E+06	798	977
2.8	1.5	3.1E+06	4.7E+06	829	1015
3.2	1.8	3.4E+06	5.0E+06	857	1050
3.6	2.0	3.6E+06	5.3E+06	883	1081
4.0	2.2	3.8E+06	5.6E+06	906	1110
4.4	2.4	3.9E+06	5.9E+06	928	1137
4.8	2.6	4.1E+06	6.2E+06	949	1162
5.2	2.9	4.3E+06	6.4E+06	968	1185
5.6	3.1	4.4E+06	6.7E+06	986	1208
6.0	3.3	4.6E+06	6.9E+06	1003	1229
6.4	3.5	4.7E+06	7.1E+06	1019	1249
6.8	3.7	4.9E+06	7.3E+06	1035	1268
7.2	4.0	5.0E+06	7.5E+06	1050	1286
7.6	4.2	5.2E+06	7.8E+06	1064	1303
8.0	4.4	5.3E+06	8.0E+06	1078	1320
8.4	4.6	5.4E+06	8.2E+06	1091	1336
8.8	4.8	5.6E+06	8.3E+06	1104	1352
9.2	5.1	5.7E+06	8.5E+06	1116	1367
9.6	5.3	5.8E+06	8.7E+06	1128	1382
10.0	5.5	5.9E+06	8.9E+06	1140	1396
10.4	5.7	6.0E+06	9.1E+06	1151	1410
10.8	5.9	6.2E+06	9.2E+06	1162	1423
11.2	6.2	6.3E+06	9.4E+06	1173	1436
11.6	6.4	6.4E+06	9.6E+06	1183	1449
12.0	6.6	6.5E+06	9.7E+06	1193	1461
12.4	6.8	6.6E+06	9.9E+06	1203	1473
12.8	7.0	6.7E+06	1.0E+07	1212	1485
13.2	7.3	6.8E+06	1.0E+07	1222	1496
13.6	7.5	6.9E+06	1.0E+07	1231	1508
14.0	7.7	7.0E+06	1.1E+07	1240	1518
14.4	7.9	7.1E+06	1.1E+07	1249	1529
14.8	8.1	7.2E+06	1.1E+07	1257	1540
15.2	8.4	7.3E+06	1.1E+07	1266	1550
15.6	8.6	7.4E+06	1.1E+07	1274	1560
16.0	8.8	7.5E+06	1.1E+07	1282	1570
16.4	9.0	7.6E+06	1.1E+07	1290	1580
16.8	9.2	7.7E+06	1.2E+07	1298	1589
17.2	9.5	7.8E+06	1.2E+07	1305	1599
17.6	9.7	7.9E+06	1.2E+07	1313	1608
18.0	9.9	8.0E+06	1.2E+07	1320	1617
18.4	10.1	8.0E+06	1.2E+07	1327	1626
18.8	10.3	8.1E+06	1.2E+07	1335	1635
19.2	10.6	8.2E+06	1.2E+07	1342	1643
19.6	10.8	8.3E+06	1.2E+07	1349	1652
20.0	11.0	8.4E+06	1.3E+07	1355	1660

1. $\sigma'm = (\sigma' + 2\sigma'k_o)/3 - k_o = 0.32$

2. $G_{max} = 1000(\sigma'm)^{0.5}k_{2,max}$

3. $V_s = ((G_{max}(g)/\gamma_{sat})^{0.5} - \gamma_{sat}=147 \text{ pcf}$

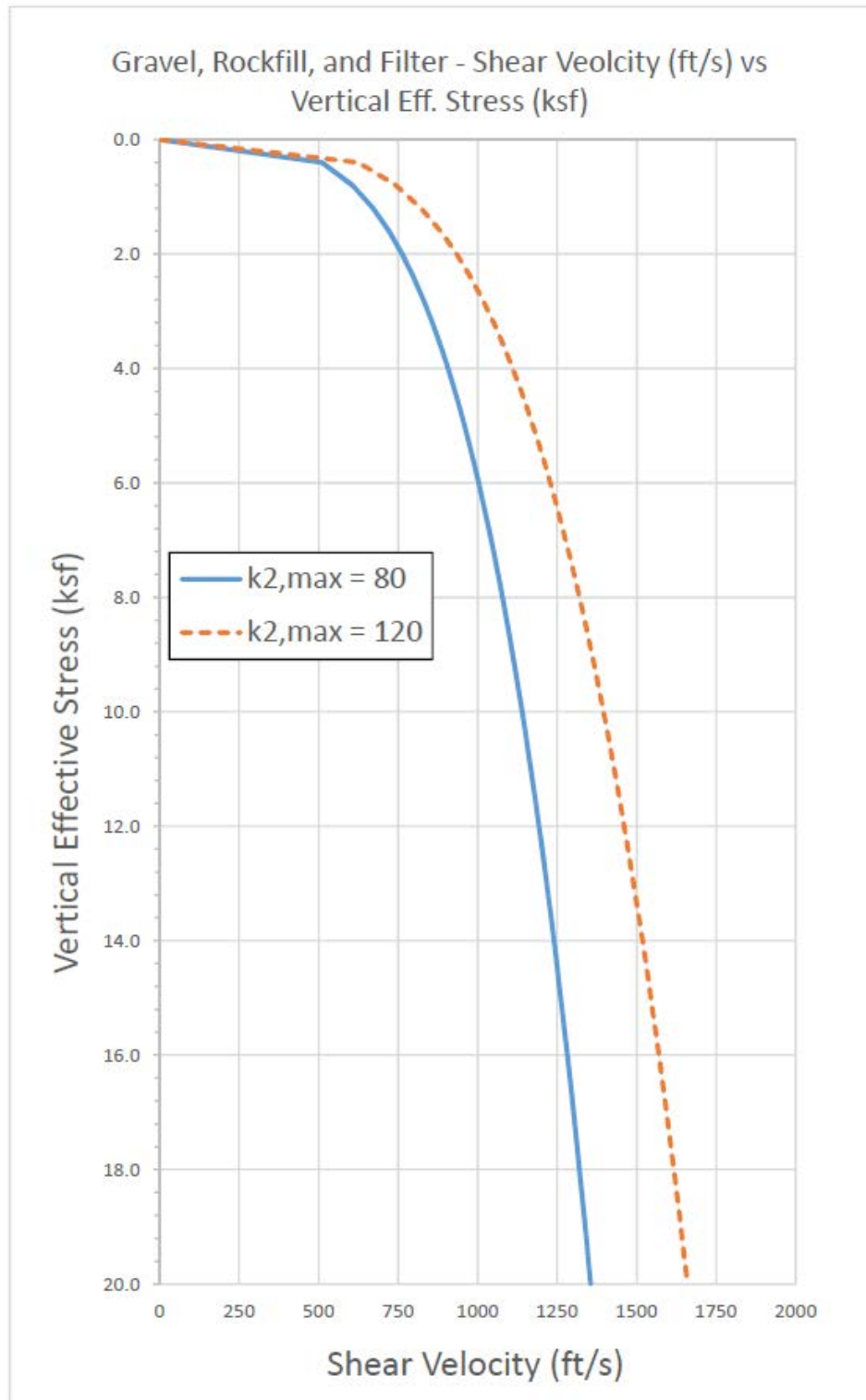


Figure 12: Gravel, Rockfill, and Filter Shear wave velocity profile – $k_{2,max}$ comparison

4.0 NON-LINEAR DEFORMATION ANALYSIS

Dynamic Analysis

The two-dimensional (2-D), non-linear embankment response was calculated using FLAC-2D. FLAC-2D is a two-dimensional numerical modeling software that uses the finite difference method to model complex soil behavior, such as problems involving large displacement or strains, and non-linear behavior (Itasca, 2016). Soil is represented by elements or zones arranged in a grid and each element responds to applied forces with respect to its assigned linear or non-linear stress/strain constitutive model. The elements yield and plastically flow, and the grid can deform and move upon loading. Input parameters for the FLAC analysis include acceleration time histories and engineering properties of the embankment materials.

4.1 Modeling sequence and Development

The embankment modeling consists of three steps outlined below.

- 1) Static analysis at the end of dam construction (built in 10 increments) to establish initial total stress state
- 2) Effective stress analysis to simulate pore and mechanical pressures from reservoir loading
- 3) Dynamic analysis to simulate response to ground motions. It should be noted that pore pressures are not generated in the Mohr-Coulomb constitutive model.

Model Setup:

The FLAC model geometry was based on the as-constructed conditions. The x and y coordinates used in the model were shown on Figure 13. The bedrock in the model was extended to a depth of 225 ft below the embankment and extended approximately 1,200 ft upstream and downstream of the embankment toes. The origin coordinate of the model (0,0) is located at the top of bedrock at the centerline of the embankment. The section analyzed is the maximum embankment section at Sta. 26+00.

In order to maintain computational accuracy of the dynamic response analysis, the element height should be less than one-eighth of a 20 Hz maximum wavelength. Assuming a bedrock shear wave velocity of 2,493 ft/s this corresponds to a maximum element size of approximately 15 ft. For the embankment, the shear wave velocity was estimated to range from 600 ft/s to 1,600 ft/s as determined

from the Gmax distribution. Based on this range of shear wave velocities, the maximum element sizes should range from 3.9 to 10.4 ft. The final selected grid size for the model was 400 by 110 with a maximum bedrock element height of 15 ft and embankment element height of 3.65 ft (Figure 14).

Boundary Conditions:

The boundary conditions used for the static and effective stress analysis were fixed in the x and y directions along the base and fixed in the x direction only along the sides of the model to allow for vertical movement in the model during loading. For the dynamic analysis, free field boundaries were assigned to the upstream and downstream bedrock edges of the model and a compliant boundary was assigned to the base using a quiet boundary in FLAC. The free field boundary is used to simulate the behavior of the extended bedrock.

Time histories:

Discussion of the time histories used in the analysis can be found in Section 2.0 Development of Seismic Design Response Spectra. The maximum duration of the selected time histories was 300 seconds. During much of the duration, the acceleration are minimal ($\pm 0.05g$). In order to speed the computational time, the records were trimmed to the duration corresponding to acceleration greater than $\pm 0.05g$. Plots illustrating the original and trimmed accelerations time histories are shown in Figure 22 and Figure 23.

Time history records are applied to the model through a shear stress applied along the base of the model. The magnitude of the shear stress is calculated through the following relationship:

$$S_{xy}(t) = -V_s \rho$$

Where: V_s = bedrock shear wave velocity

ρ = bedrock density (γ/g)

Static Analysis:

The initial static analysis consisted of incrementally building the embankment in 10 segments to the full height of 347 ft. The analysis was solved after each increment and any deformations set to zero before applying the next increment. This initial stress state was used in the effective stress analysis. Initial static stress states are shown on Figure 15 and Figure 16.

Seepage Analysis:

The embankment was analyzed under normal full pool conditions. A phreatic surface was estimated and applied to model in order to determine the pore pressure distribution through the model (Figure 17). After the model came to equilibrium, displacements and velocities were set to zero and the mechanical pressure of the reservoir was applied embankment and model was brought to equilibrium. This state represents the initial stress state prior to dynamic loading. The final stress and strain states prior to dynamic loading are shown in Figure 18 through Figure 20.

Dynamic Analysis:

Prior to dynamic loading, the shear modulus properties were updated to account for mean effective stress (**Figure 21**). The time history from the crustal source was used in the analysis. Frequency dependent Raleigh damping of one percent was added to the model to match closely to assumed natural frequency of the embankment.

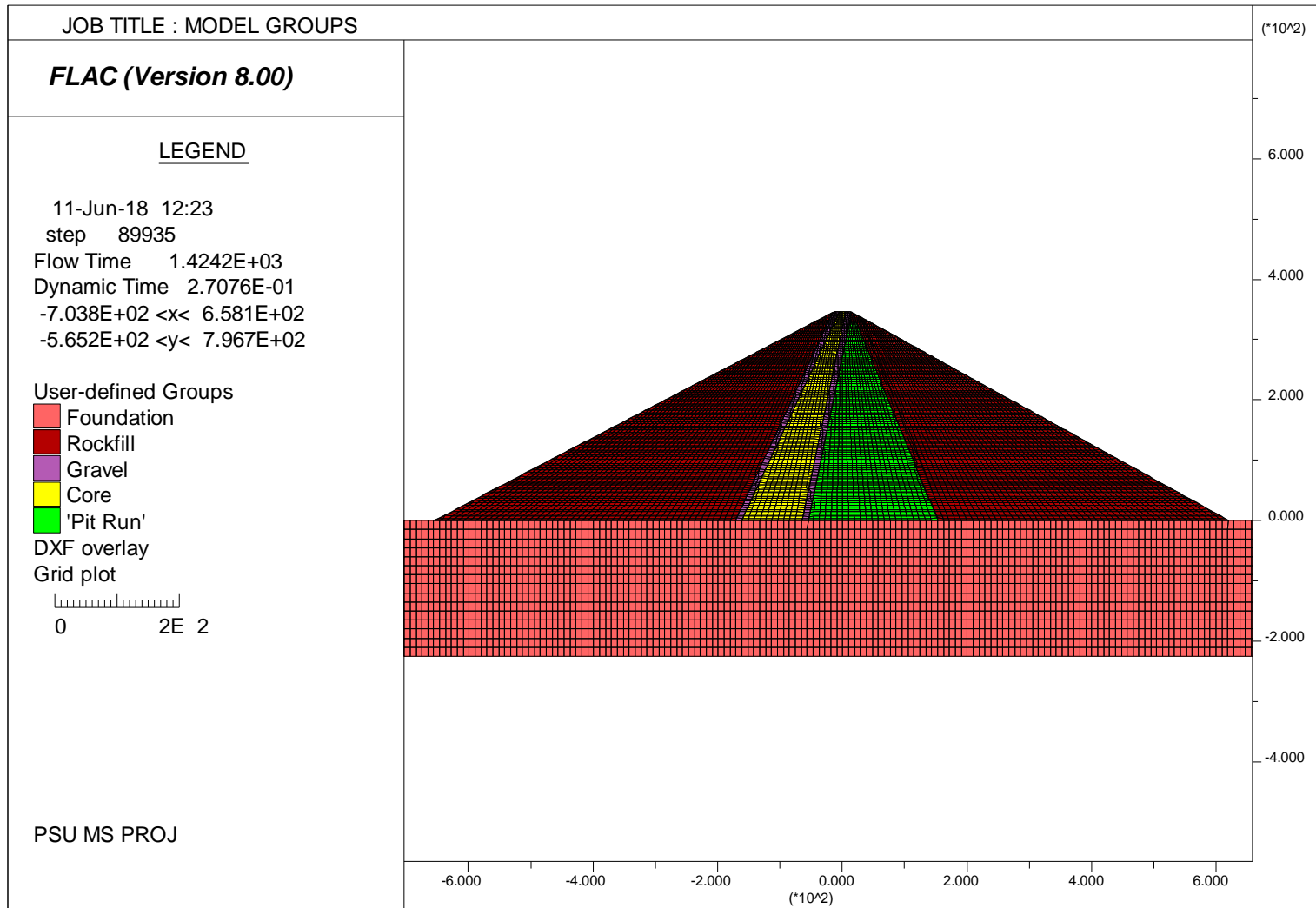


Figure 14: FLAC Model – Embankment Groups and Mesh

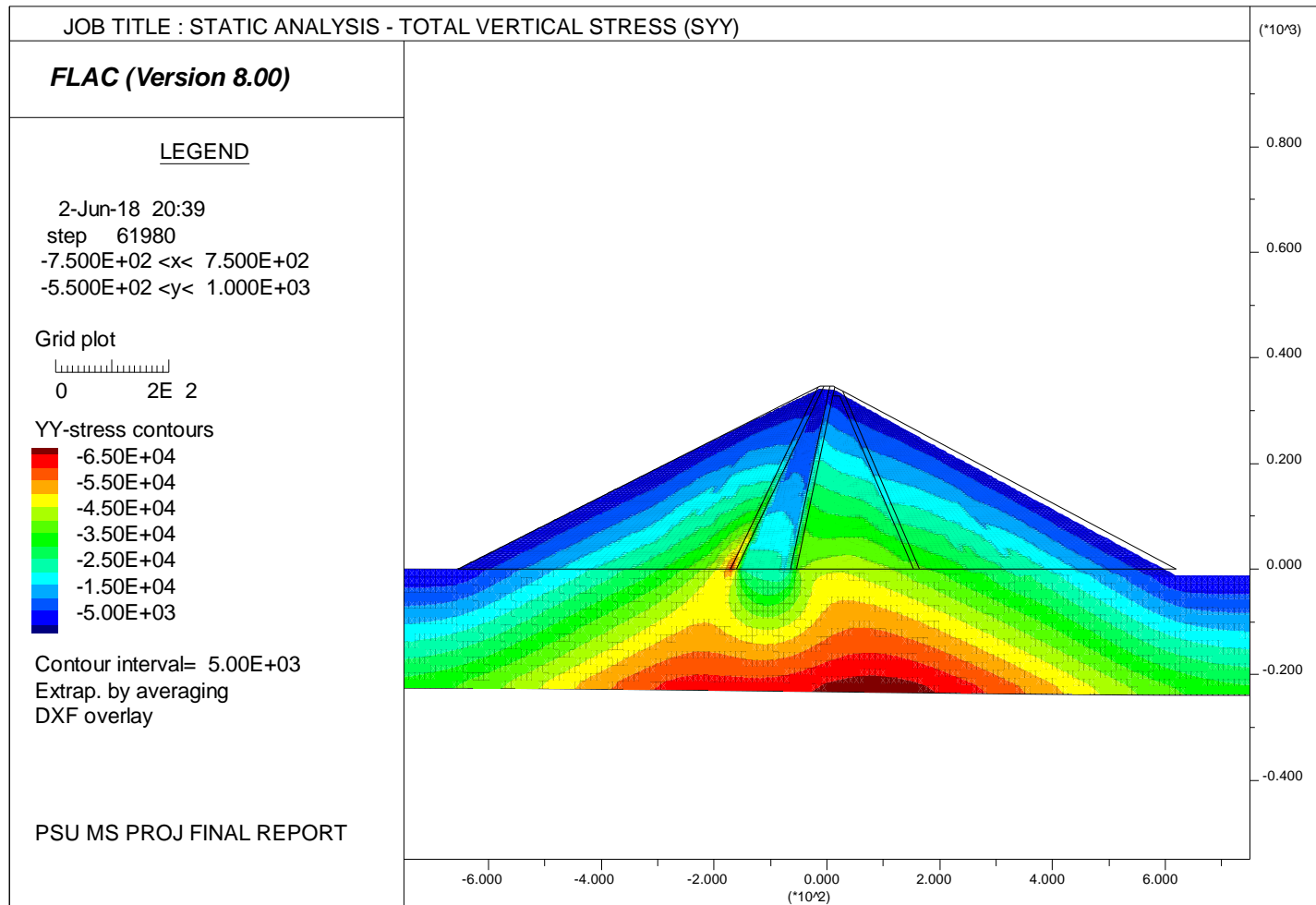


Figure 15: Static Analysis – Total Vertical Stress Contours (SYY)

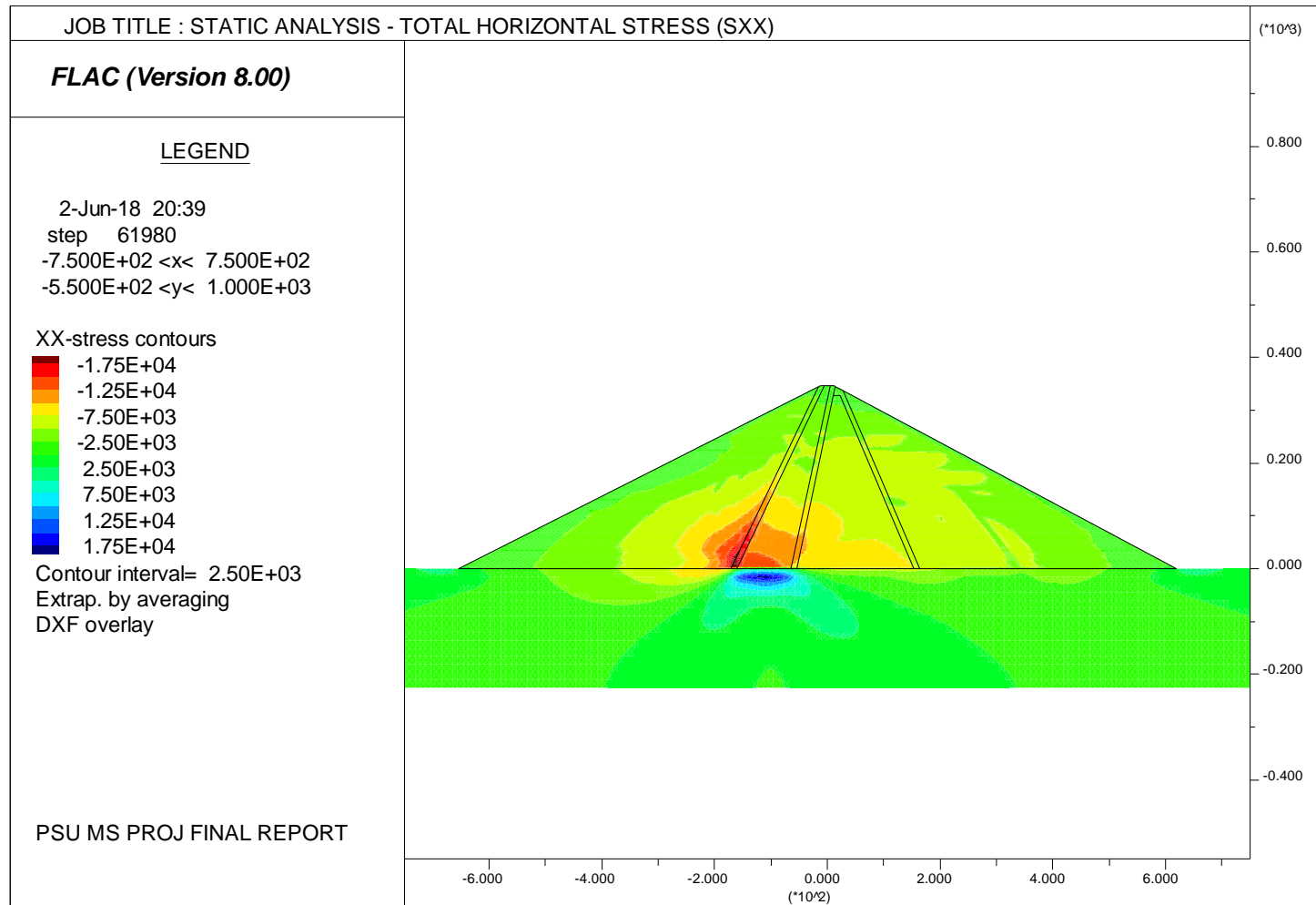


Figure 16: Static Analysis – Total Horizontal Stress Contours (SXX)

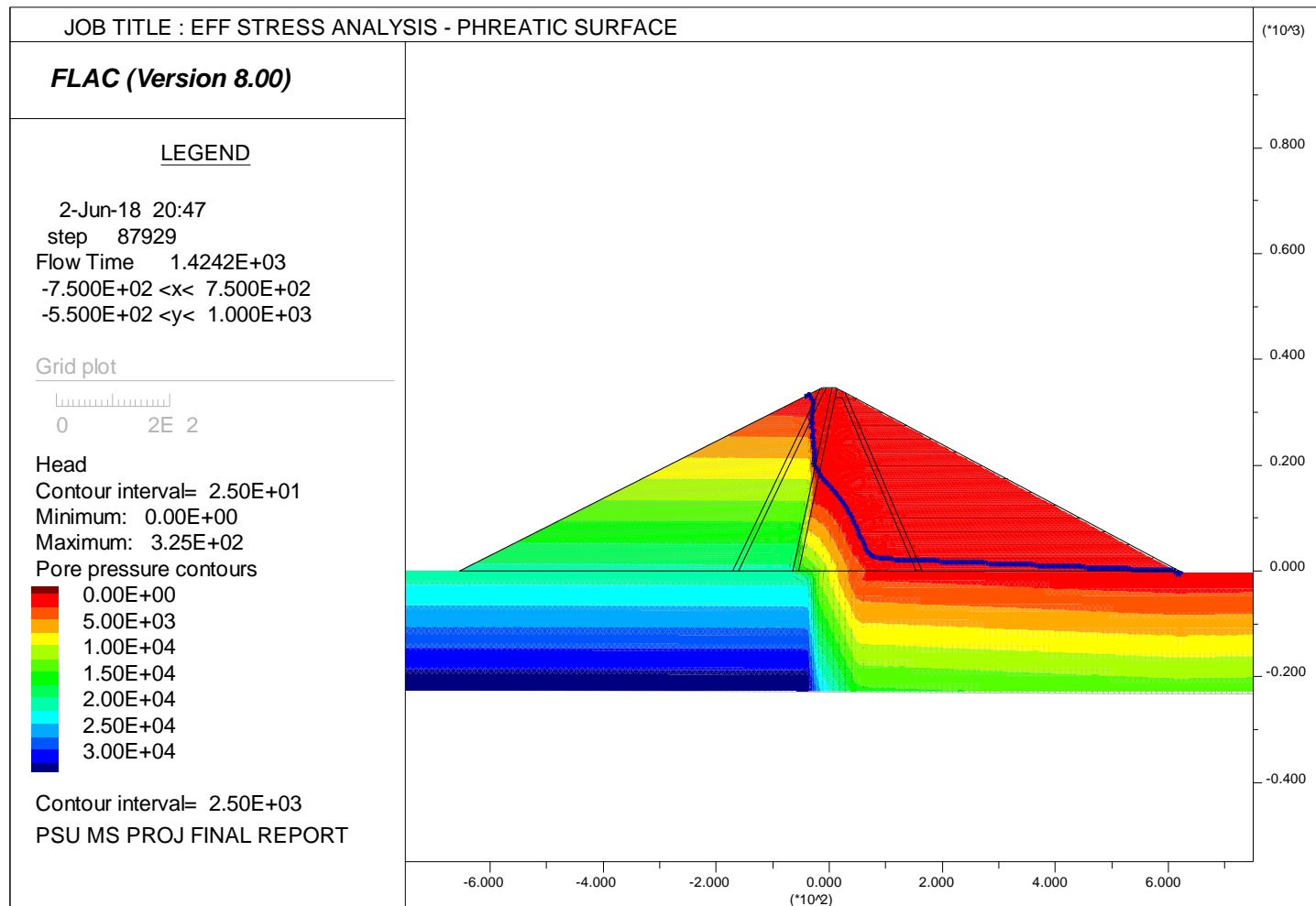


Figure 17: Seepage Analysis – Phreatic Surface (blue line) and Pore Pressure Contours

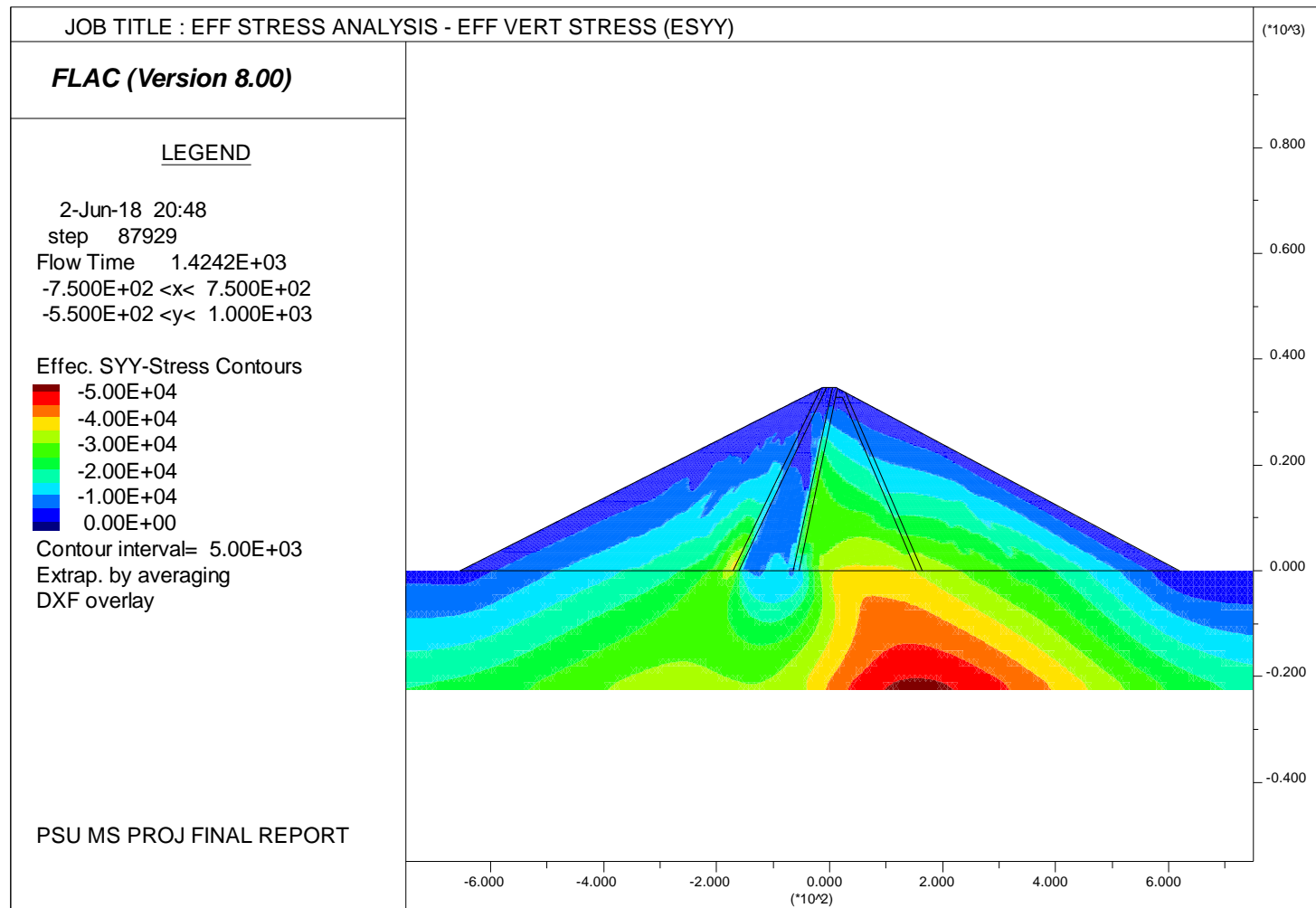


Figure 18: Seepage Analysis – Effective Vertical Stress Contours (ESYY)

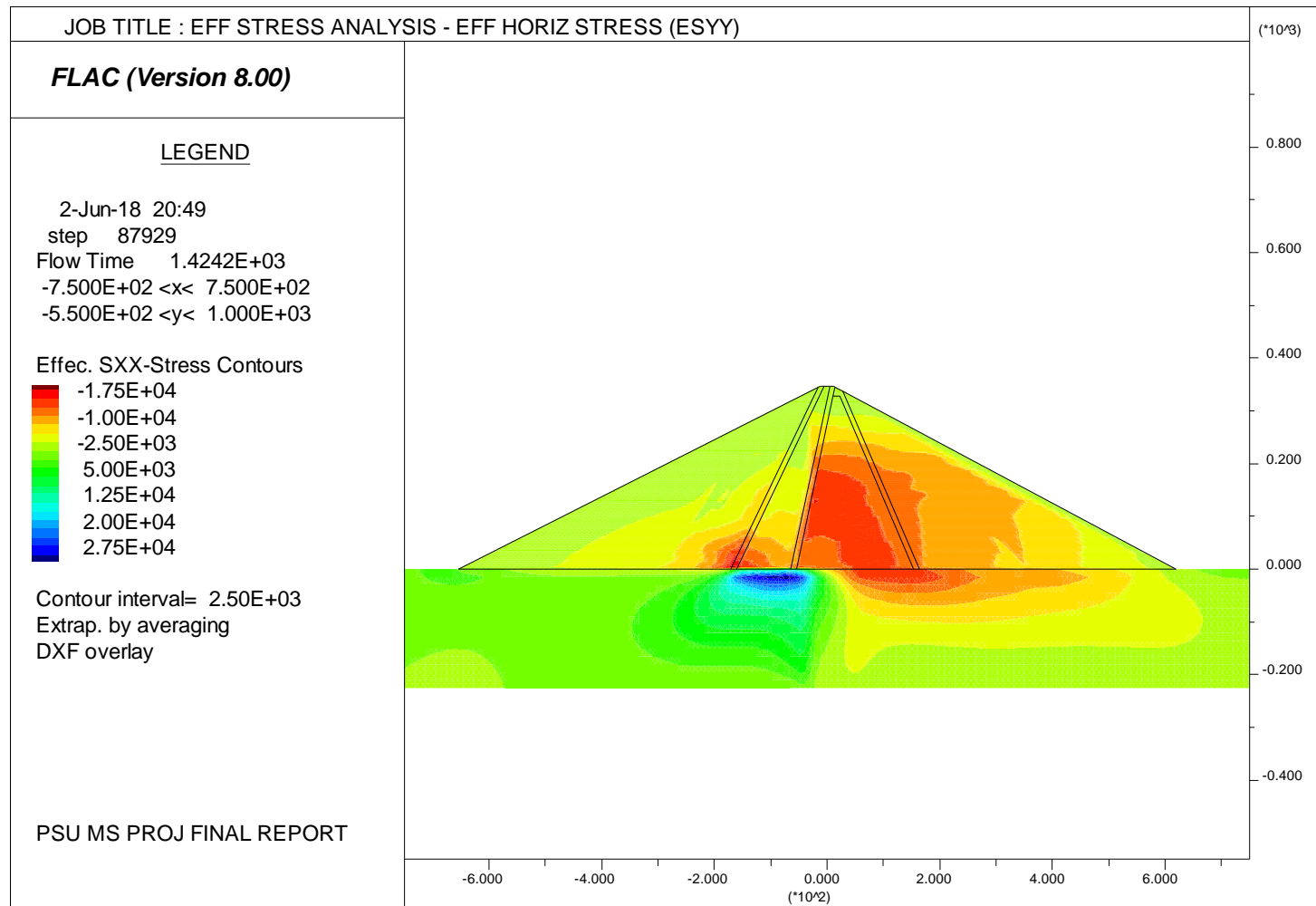


Figure 19: Seepage Analysis – Effective Horizontal Stress Contours (ESXX)

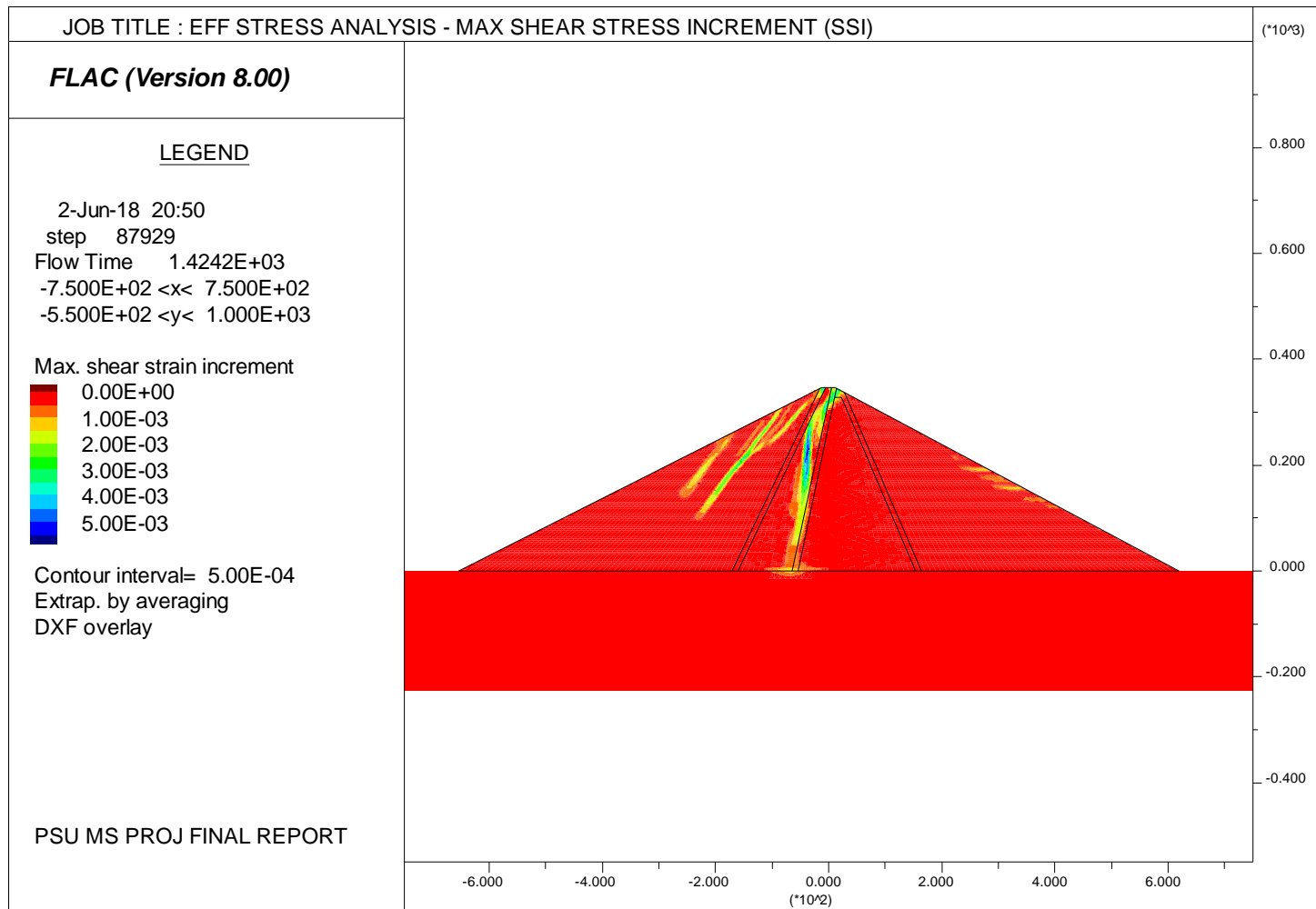


Figure 20: Maximum Shear Strain Increment pre Dynamic Loading (SSI)

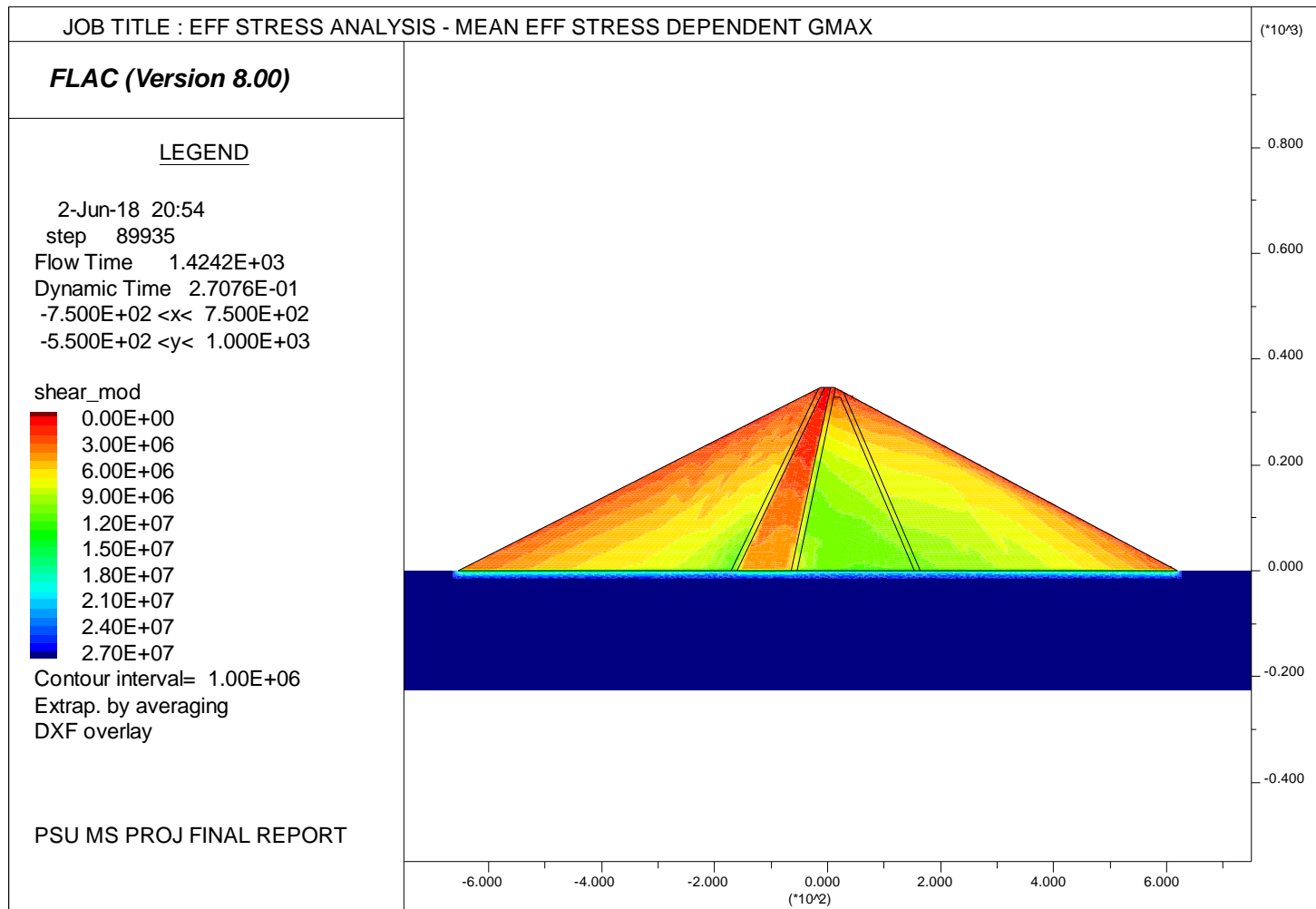


Figure 21: Mean Effective Stress Dependent Gmax

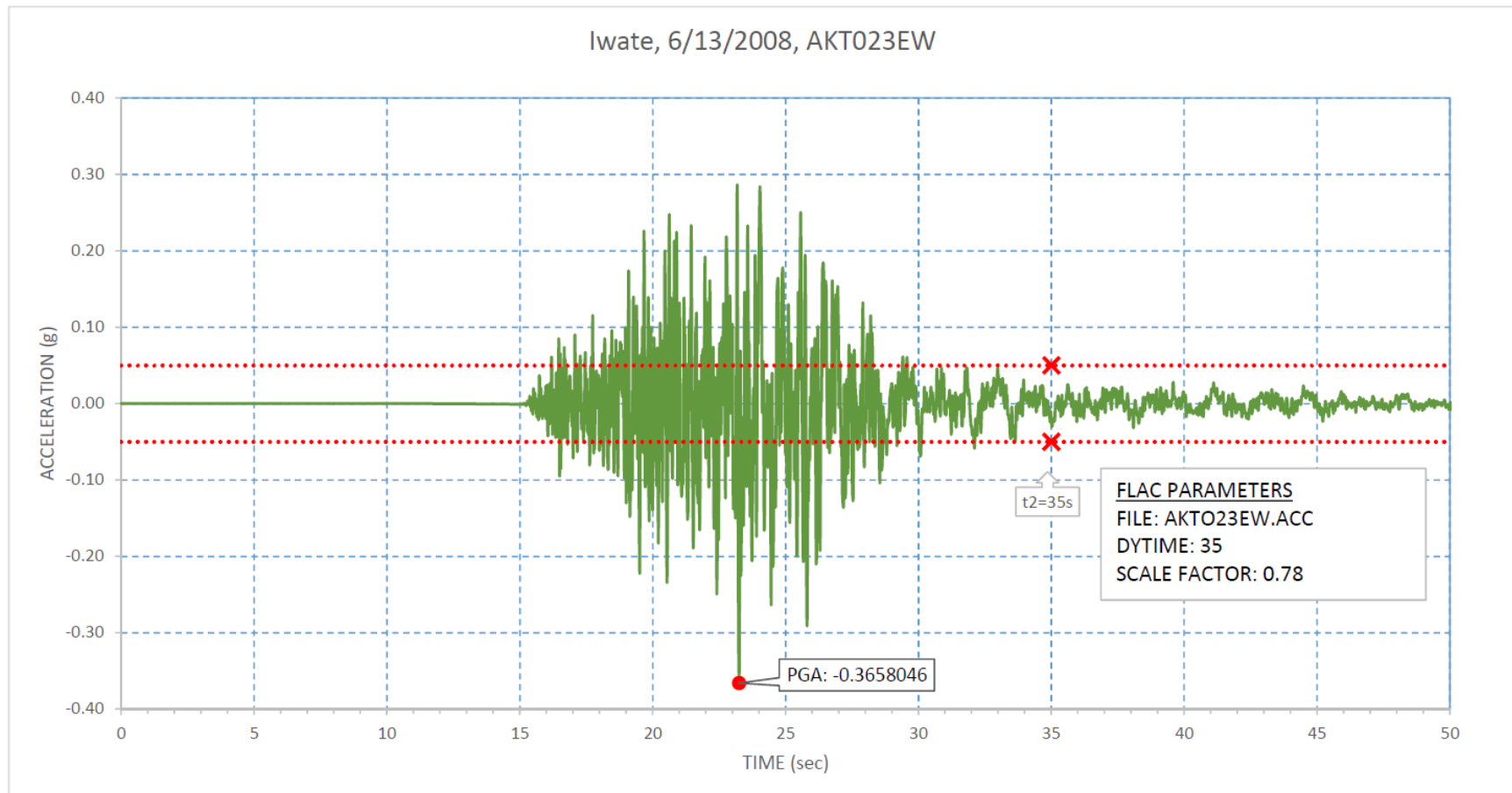


Figure 22: Acceleration Time History (AKT023EW)

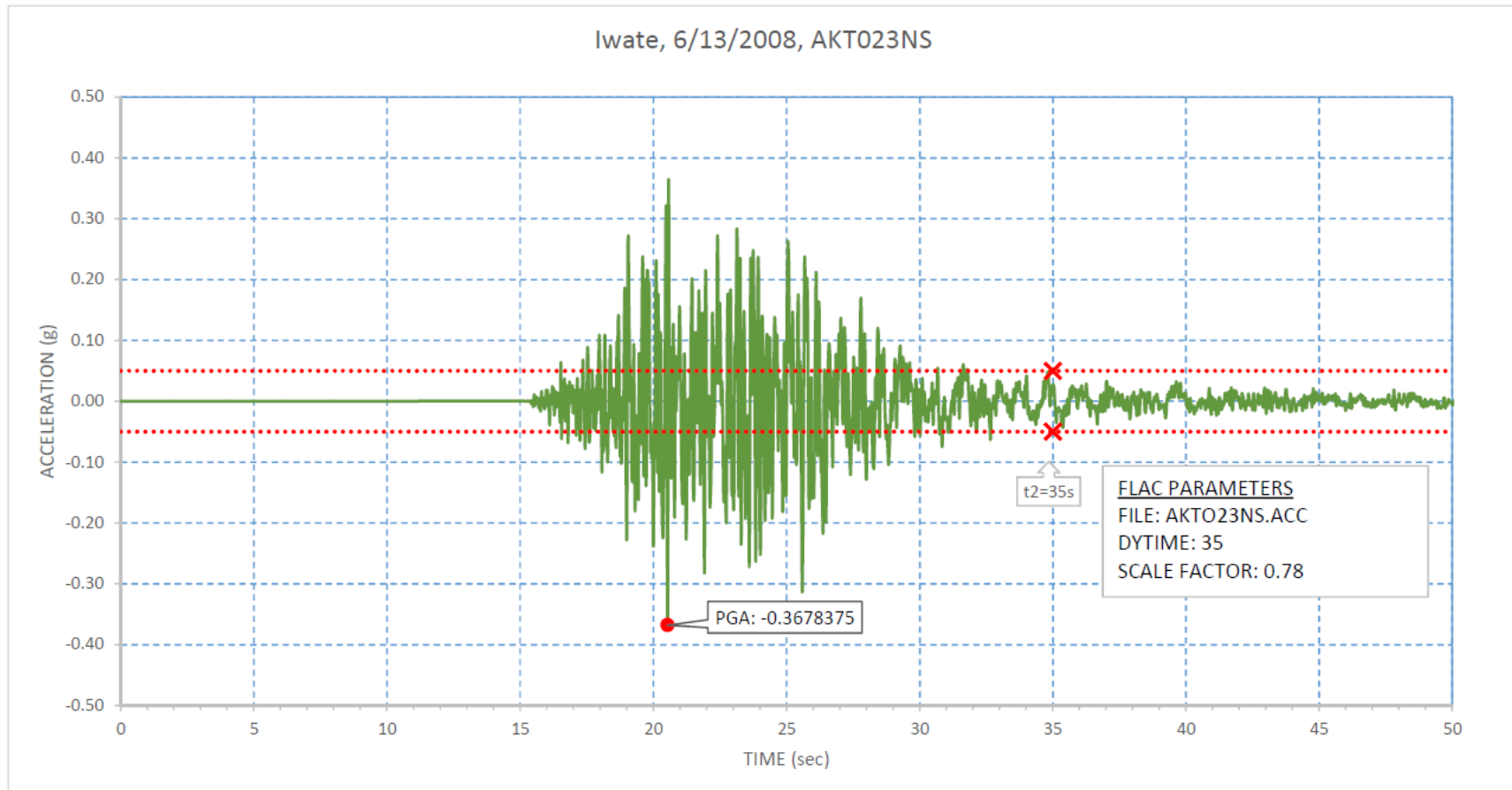


Figure 23: Acceleration Time History (AKT023NS)

4.2 Representative Results

For the purposes of this project, only the crustal source with two horizontal components (AKT023EW and AKT023NS) were applied to the model. Representative FLAC output plots for record AKT023EW are shown below. Vertical, horizontal, and overall displacement contours are shown on Figure 24 through Figure 26. Shear strains are not directly calculated in FLAC but the shear strain increment (SSI) variable can be used as an analog to shear strain. SSI contour plots at the end of dynamic loading are shown on Figure 20. Vertical displacements at the crest and top of bedrock were tracked through the time history and are shown on Figure 28. An exaggerated deformed mesh is shown in Figure 29.

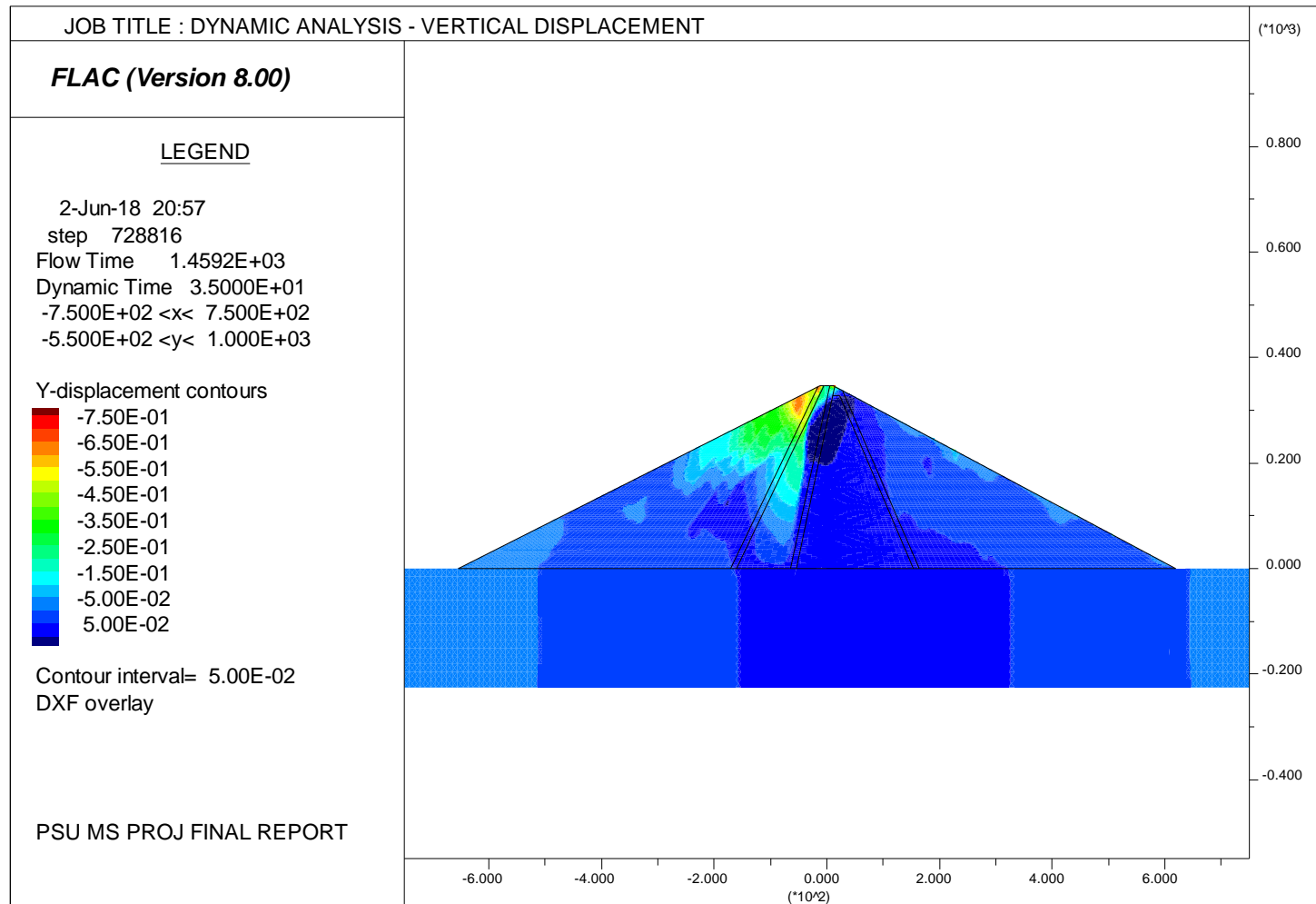


Figure 24: Dynamic Analysis – Vertical Displacement Contours (AKT023EW)

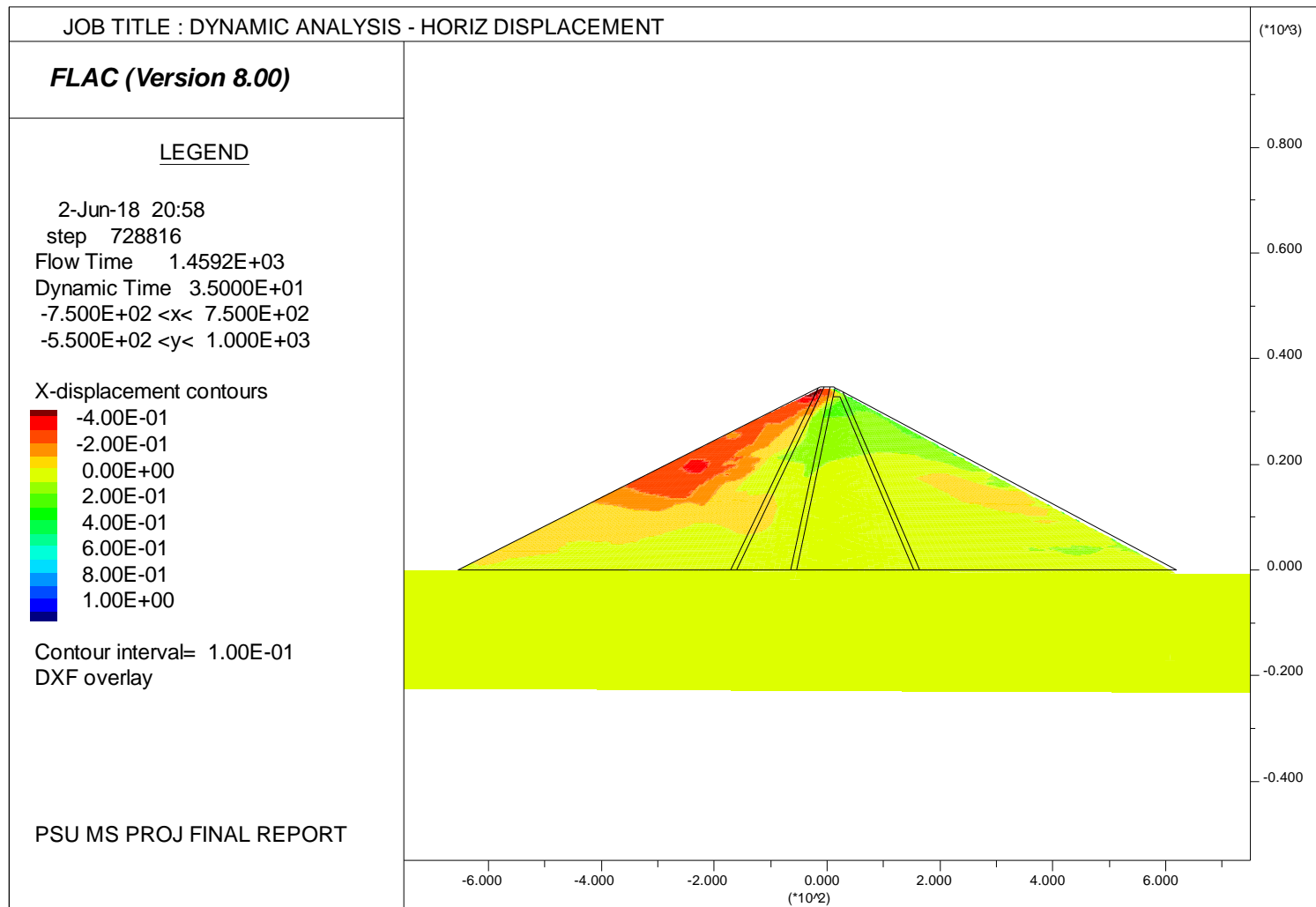


Figure 25: Dynamic Analysis – Horizontal Displacement Contours (AKT023EW)

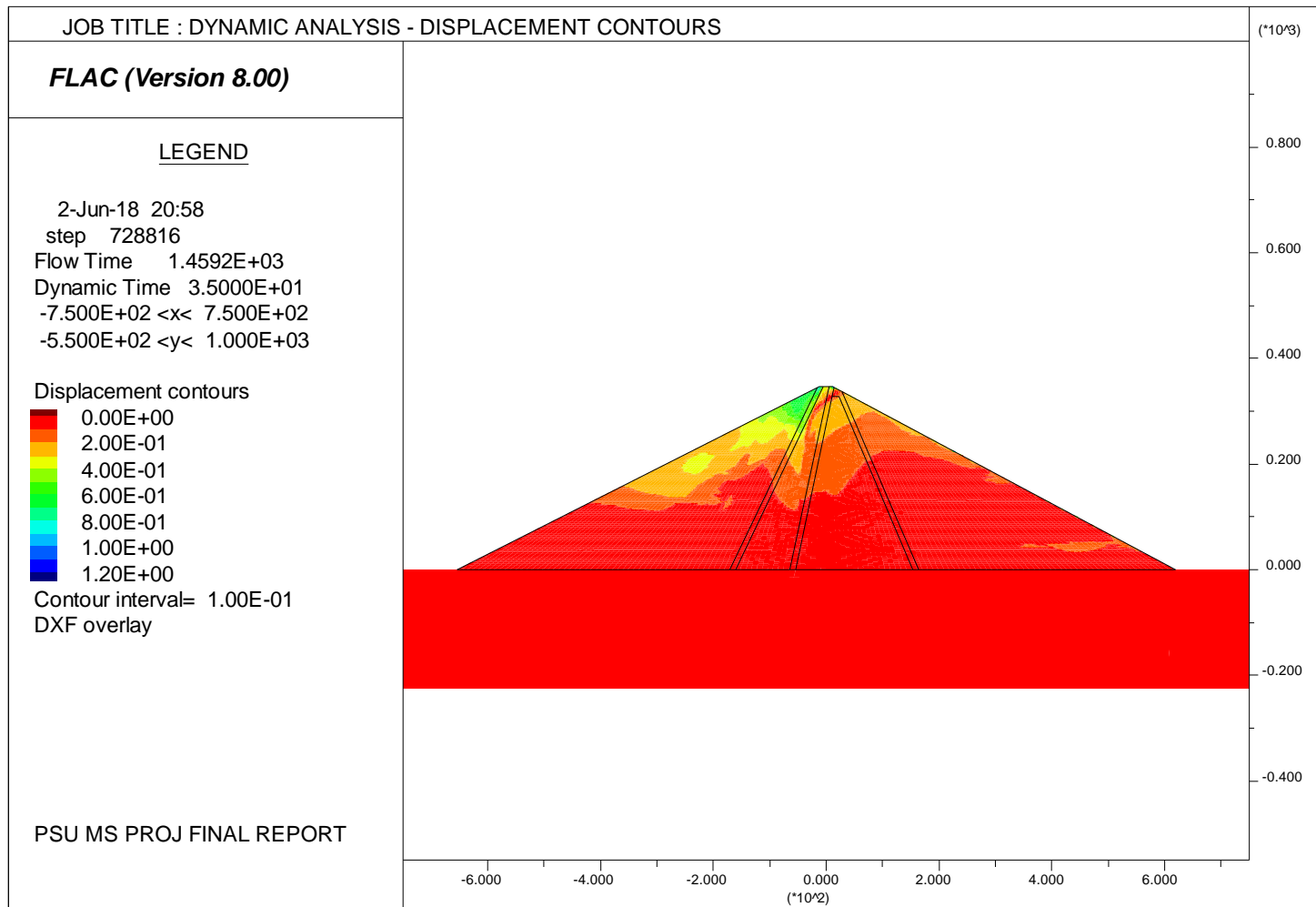


Figure 26: Dynamic Analysis – Displacement Contours (AKT023EW)

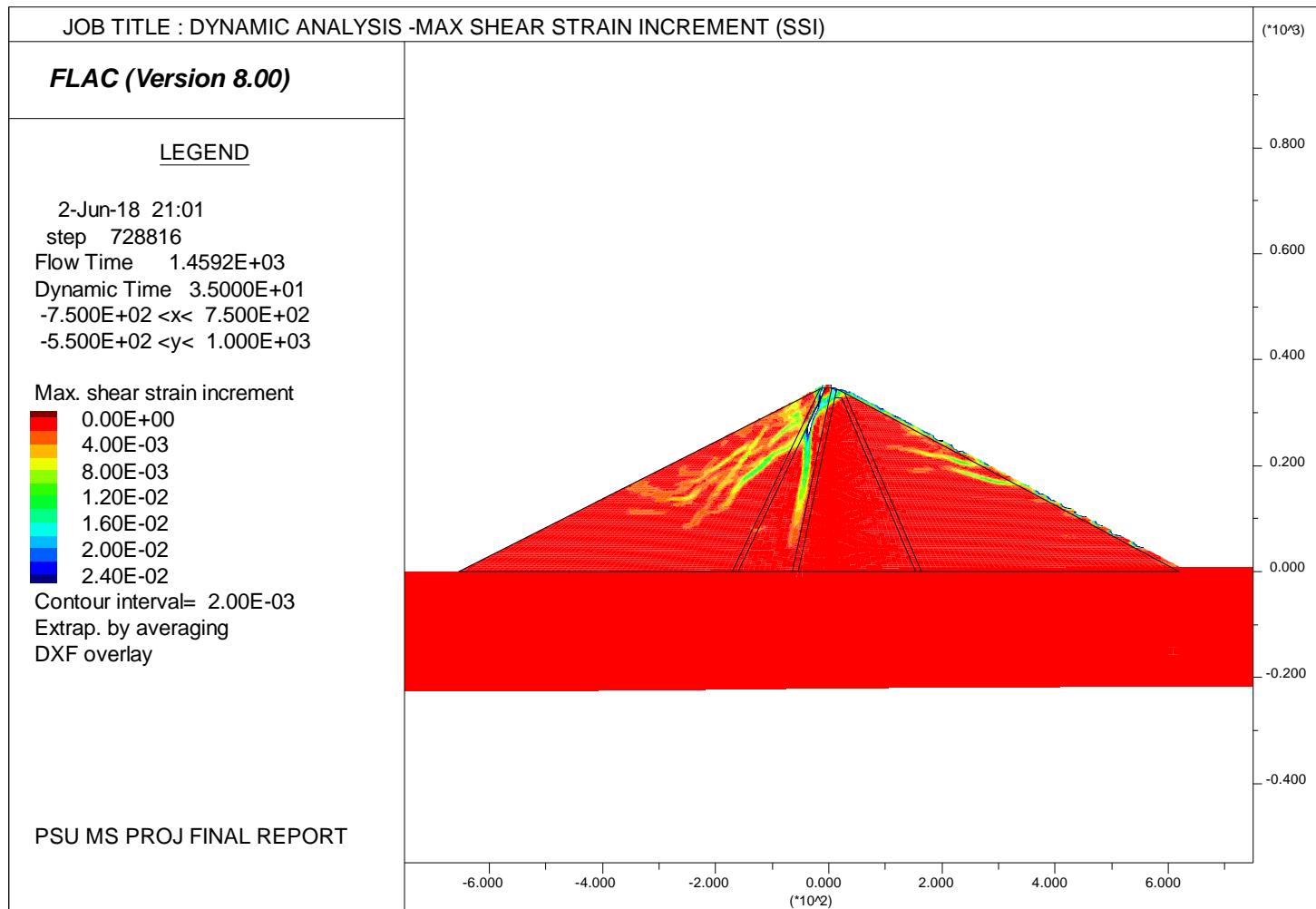


Figure 27: Dynamic Analysis – Maximum Shear Strain Increment at end of Dynamic Loading (SSI) (AKT023EW)

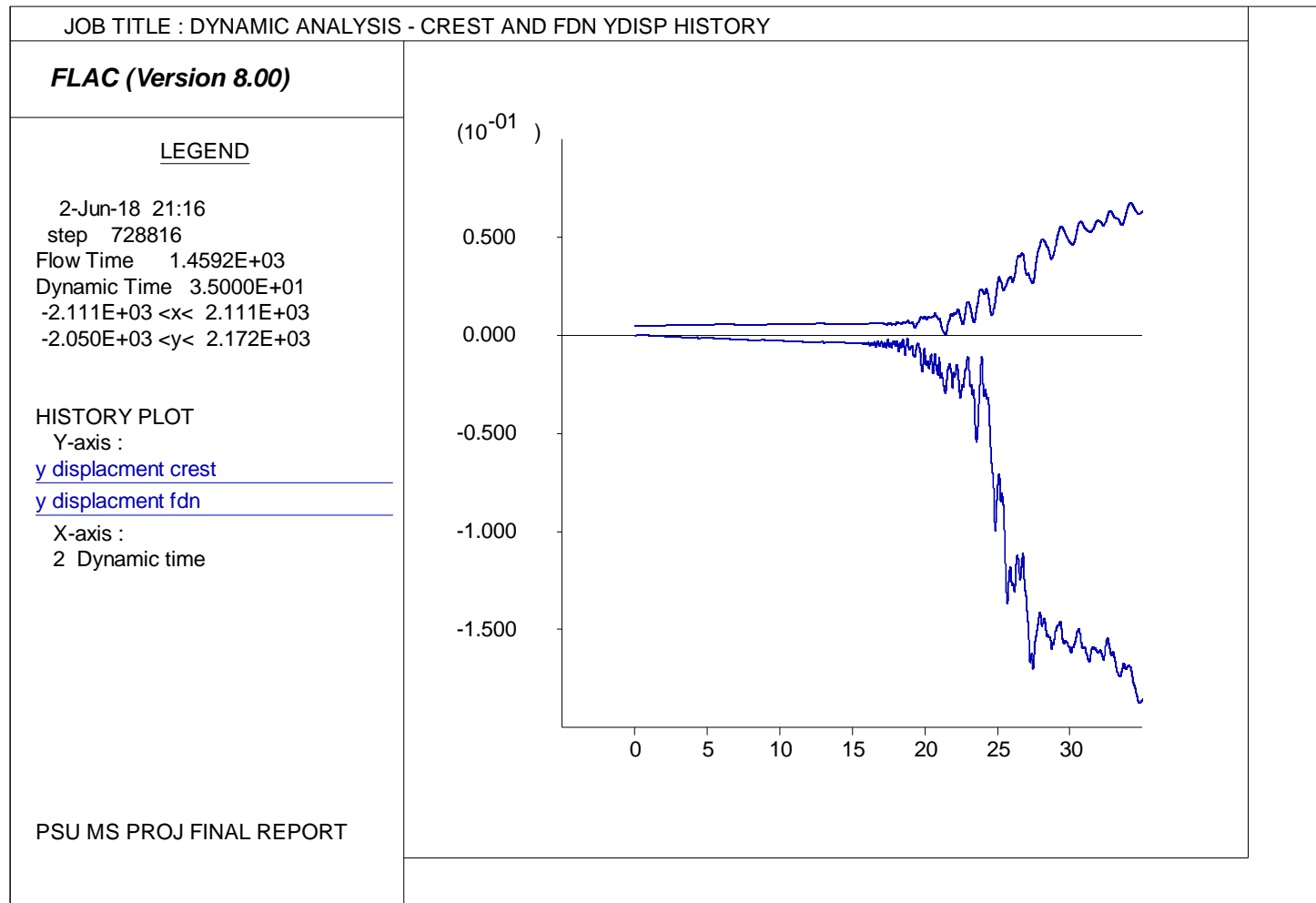


Figure 28: Dynamic Analysis – Crest and Top of Bedrock Vertical Displacement History (AKT023EW)

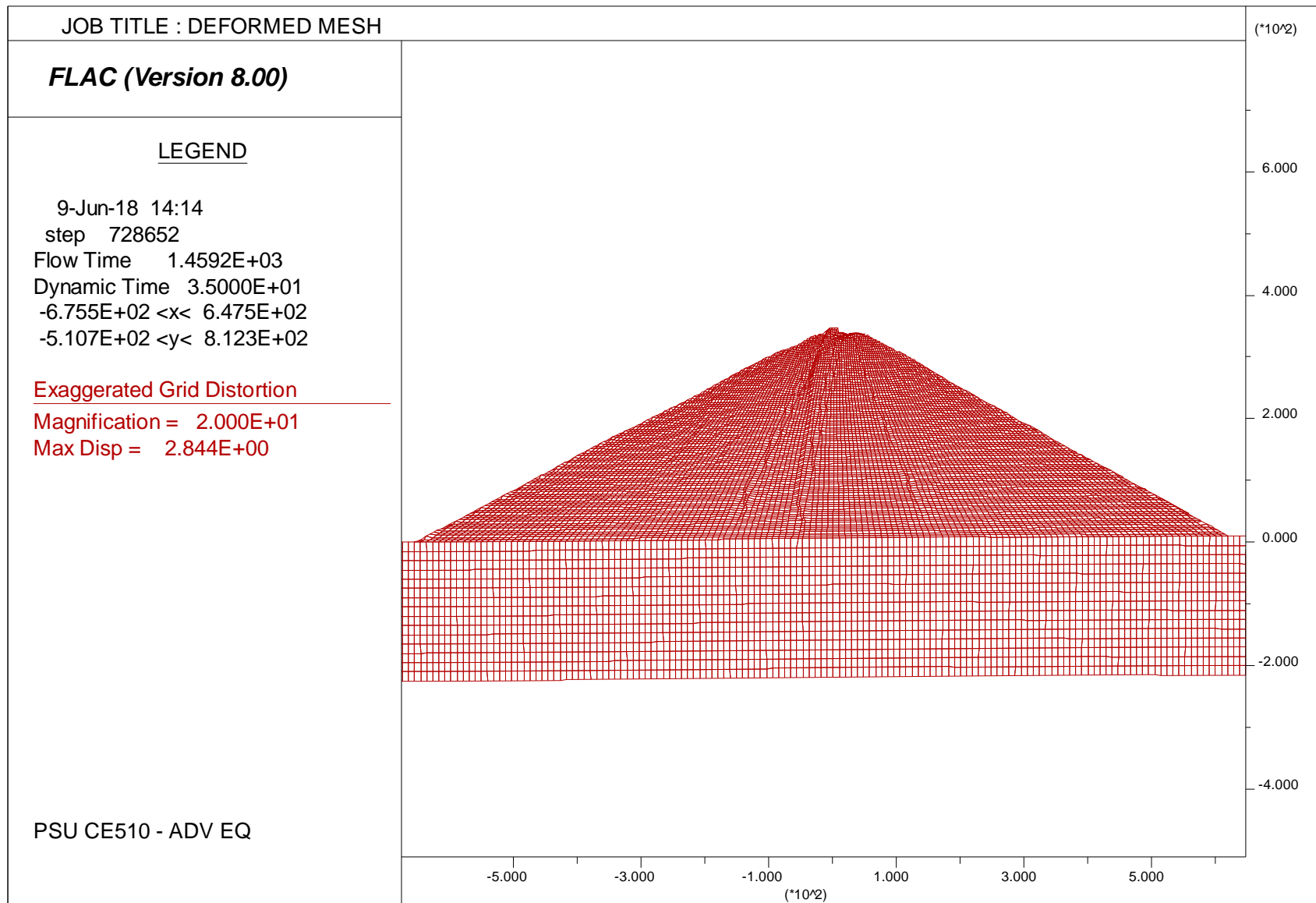


Figure 29: Dynamic Analysis – Exaggerated Deformed Mesh (AKT023EW)

4.3 Results

The estimated maximum and permanent vertical and horizontal deformations for records AKT023ES and AKT023NS are shown in Table 9. Time history plots of the horizontal and vertical displacements are shown in Figure 30 and Figure 31. Results from the crustal sources were compared to the estimated deformation predicted by the Swaisgood (2014) regression equation. The regression equation relates crest deformation to earthquake magnitude, peak ground acceleration, and height of embankment and foundation for cases without significant liquefaction induced deformations. The equation was developed in 2003 and was updated in 2014 to include case histories data from the 2011 Tohoku (Mw 9) and 2010 Maule (Mw 8.8) subduction earthquakes. The median predicted deformation from the Swaisgood (2014) equation is 0.27 ft which compares well to the permanent relative vertical deformation predicted by FLAC (Figure 32).

Table 9: Deformation Results

Record	Permanent Crest Deformations (ft)		Maximum Crest Deformations (ft)	
	Horizontal	Vertical	Horizontal	Vertical
AKT023EW	-0.03	-0.26	-0.33	-0.26
AKT023NS	-0.25	-0.31	-0.39	-0.31
Mean	-0.14	-0.29	-0.36	-0.29

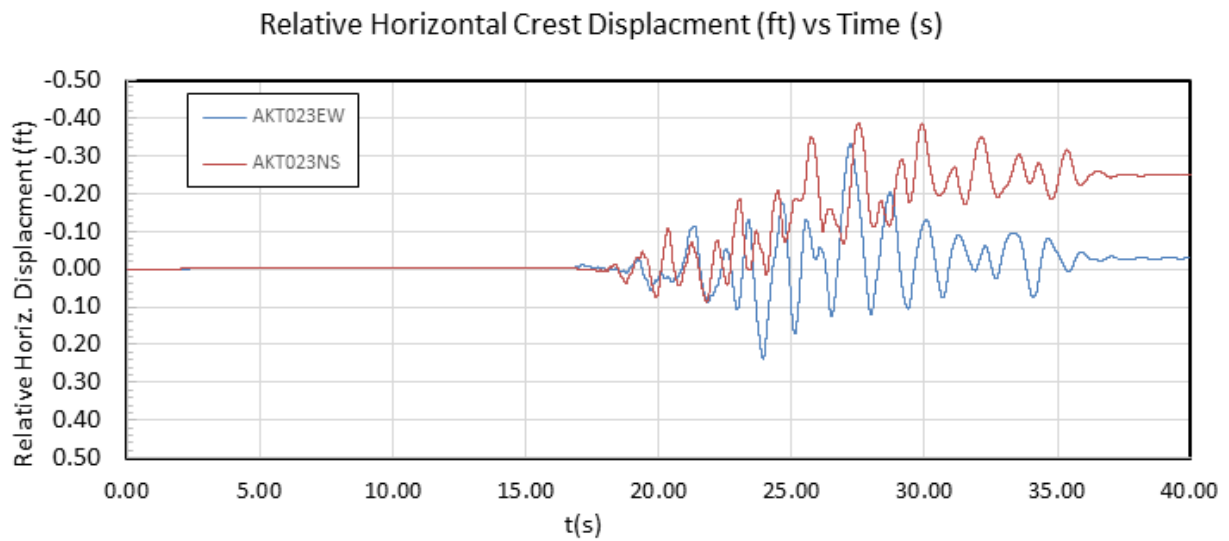


Figure 30: Relative Horizontal Crest Displacement (ft) versus Time (s) for AKT023EW and AKT023NS

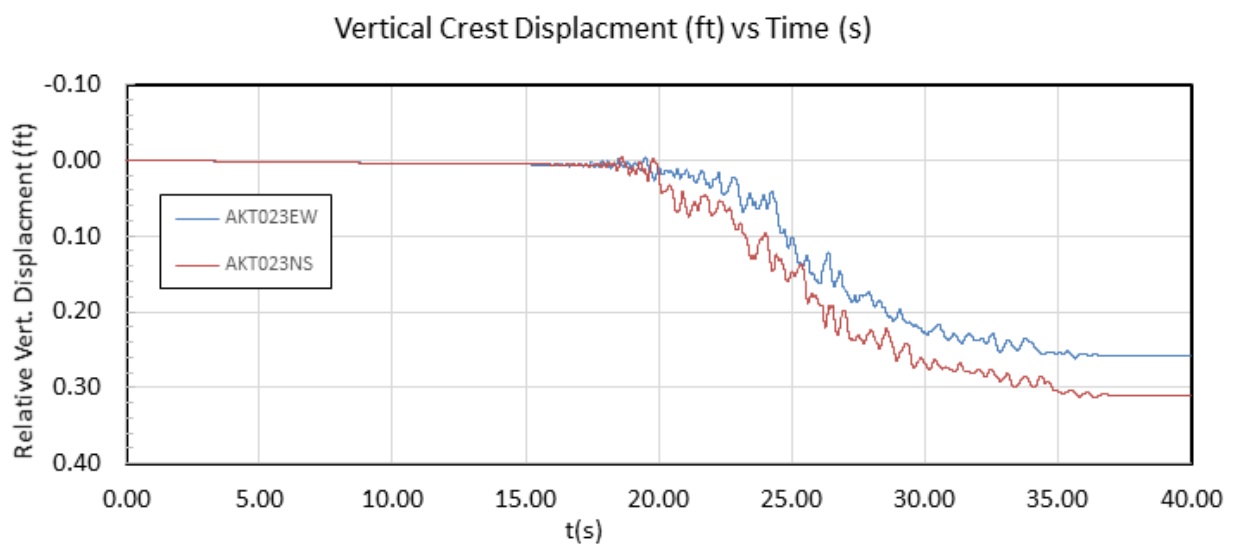


Figure 31: Vertical Crest Displacement (ft) versus Time (s) for AKT023EW and AKT023NS

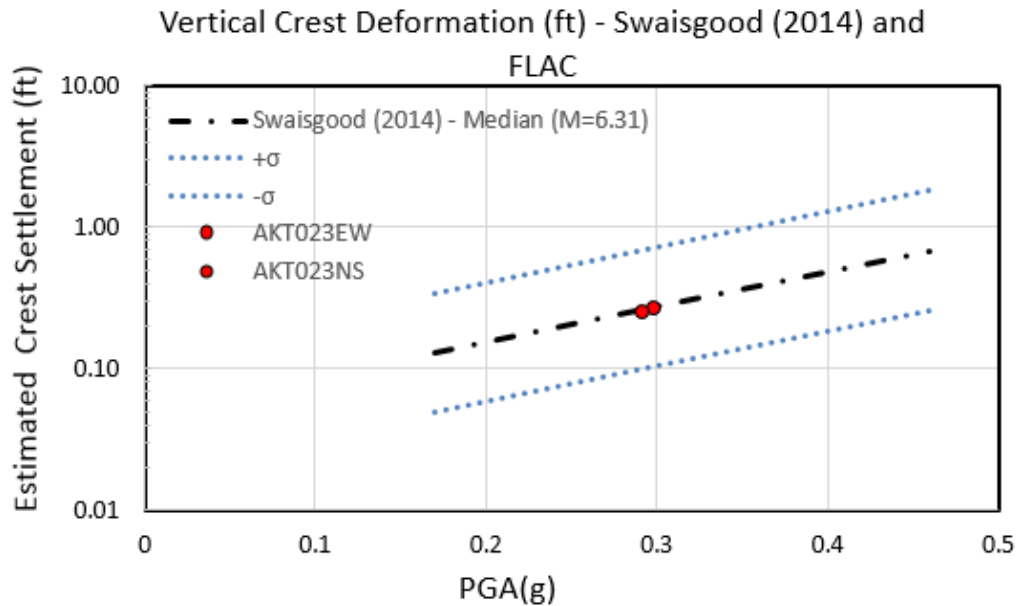


Figure 32: Vertical Crest Deformation Comparison – Swaisgood (2014) and FLAC vs PGA (g)

Bedrock input PGA and the maximum PGA calculated at the crest are shown in Table 10 and Figure 33. The amplification factors at PGA were compared to a relationship between the input and crest PGA established by Yu et al. (2012). The results are plotted on Figure 16 and the FLAC results compare well to the relationship and are considered reasonable. Amplifications at all periods are shown in Figure 35.

Table 10: Input and Crest PGAs, and Amplification Factors

Record	PGA (Input)	PGA (Crest)	PGA Amplification Factor
AKT023EW	0.298	0.662	2.221
AKT023NS	0.292	0.415	1.421

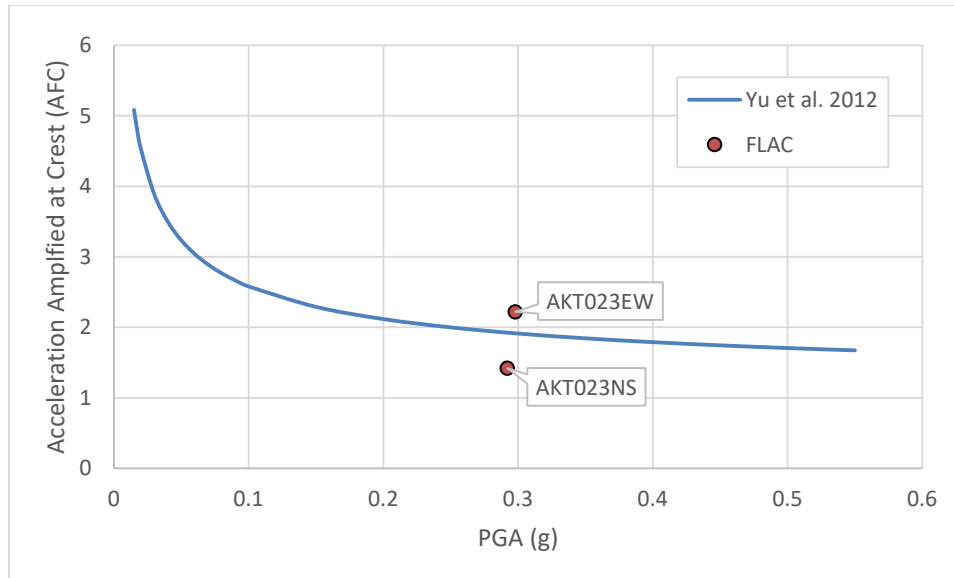


Figure 33: PGA vs Amplification Factor at Crest – Yu et al. 2012 and FLAC results

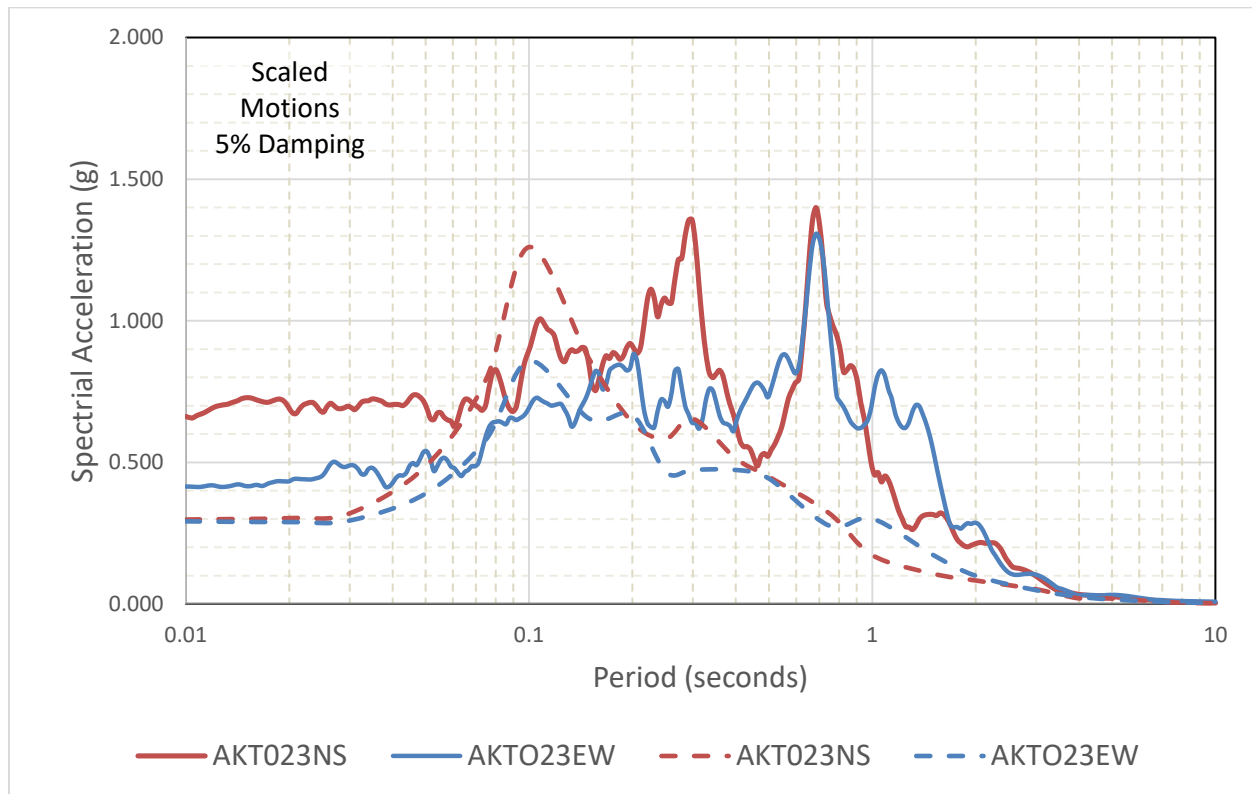


Figure 34: Crustal Source Response Spectra – Input Outcrop Motion (dashed) and Top of Crest (solid)

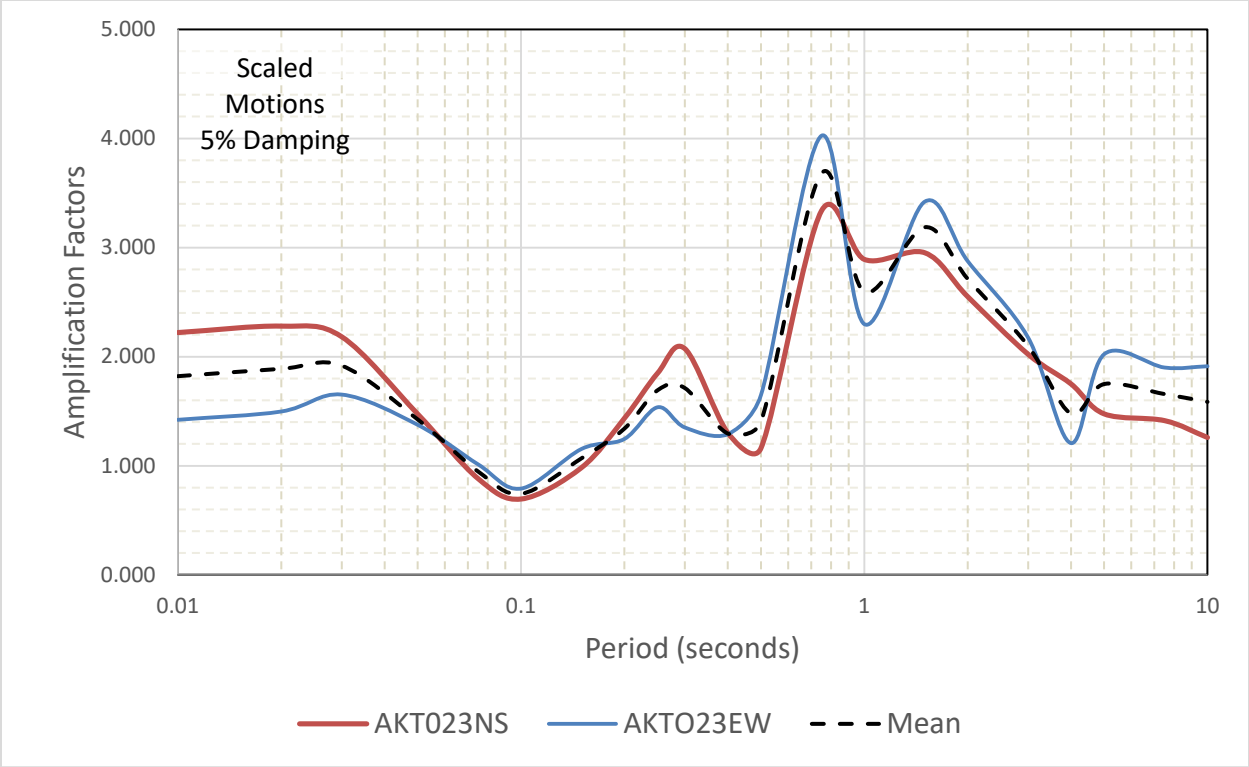


Figure 35: Crustal Source Amplification Factors – Input Outcrop Motion relative to Top of Crest

5.0 CONCLUSIONS AND OBSERVATIONS

This objective of this study was to conduct a non-linear deformation analysis (NDA) of an embankment dam subject to seismic loading using the NDA program FLAC. The results from the crustal ground motion records indicate that the permanent vertical crest deformation compares well to the estimated deformation from Swaisgood (2014) and the bedrock to crest PGA amplifications factors appear to be reasonable when compared to published relationships. The shear strain patterns in the model indicate that failure surfaces are developing down through the core, extending laterally through the upstream shell but that fully formed failure surfaces have not developed

The following are observations of the analysis process that would be beneficial to consider in future analyses:

Modeling: The crustal sources used in this project had relatively short durations (35s) compared to the subduction zone sources (150-300s). The longer duration time histories will require significantly more computational time. In order to speed computational time and allow for the usage extra variables to be calculated while the model is solving, the size of the model should be reduced to as small as is reasonable to maintain computational accuracy and to best approximate the geometry of key embankment features.

Parametric Studies: The model developed represents an idealized representation of the embankment. Selected embankment material parameters were assumed from design parameters. These parameters would likely vary throughout the embankment which may change the predicted response of the embankment. Therefore it is important to conduct parametric studies in order to capture the range of predicted embankment response to seismic loading.

Initial Stress State: The goal of conducting the initial static stress analysis was to establish a reasonable initial stress distribution that conformed to the expected behavior of the embankment dam. FLAC has the capability to perform a transient seepage analysis that can be used to estimate the pore pressure distribution throughout the model. However, convergence issues occurred when attempting to model the pore pressure distribution in this manner. Therefore, for simplicity, a phreatic surface was assumed. The phreatic surface assumed or modeled should ideally be calibrated to observed

measurements. Additionally, it would be beneficial to calibrate the initial stress model to observed historical deformation.

Constitutive Model: The constitutive model used in the analysis was a simple Mohr-Coulomb model with sigmoidal 3 function used to approximate the G/G_{max} curves. It is a simple model, suitable to this level of analysis but it has some drawbacks. It is not capable of pore pressure generation, stress dependence is not built into the model (but could be implemented), and dilation angle was not included. Future modeling efforts should consider using other constitutive models in addition to the Mohr-Coulomb model in order to better understand the range of embankment response.

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7.0 APPENDIX – FLAC CODE

```

1 ;Project Record Tree export
2 ;File:E:\PSU\Final Project\Production Runs\MS PROJ DAM X CODE.dat
3 ;Units: Imperial: foot-slug-second;Date: Jun 11, 2018 4:11:34 PM
4 ;Title: PSU MS PROJ
5 ;Branch 1:GEO.sav
6 ; Silas T. Sanderson, PSU MS Project
7 ; Original Date: 14 May 18
8 ; Model: Dam X
9 ; Purpose: configure geometry, load dynamically, assess total vert deformation and
embankment response
10 ; Units: ft, s, slugs
11 config ats gwflow dynamic extra 20 ;configures model for seepage flow, dynamic
analysis, and 20 extra variables
12 set flow off dyn off ; set seepage (flow) and dyanmic loading off for intial static
analysis
13 set grav = 32.2 ;gravity 32.2 ft/s/s
14 water bulk = 0.0 ; use 0.0 for initial drained static analysis
15 water dens = 1.9379 ; water density in slugs/ft3
16 ;-----
17 ;STEP 1 - IMPORT GEOMETRY
18 ;-----
19 grid 400, 110 ; define grid for entire model 3800 ft x 397 ft each grid is 5 ft x 5 ft
20 def model_props ;useful routine to quickly call i,j limits
21 $JGPMAX = jgp ;111
22 $IGPMAX = igp; 401
23 $IGPMIN = 1
24 $JZMAX = jzones ; 110
25 $IZMAX = 1;
26 end
27 model_props
28 gen -1900.0,0.0 -400.0,347.0 400.0,347.0 1900.0,0.0 i=1,401 j=16,111 ; generate
embankment elements as a rectangle - element height = 347 ft/110 = 3.15 ft
29 gen -1900.0,-225.0 -1900.0,0.0 1900.0,0.0 1900.0,-225.0 i=1,401 j=1,16 ; generate
bedrock as a rectangle - element height = 225 ft/15 = 15 ft
30 m mohr
31 table 1 -1900.0,0 -653.95,0 -12,347 12,347 619.25,0 1900.0,0 ; top ground surface of
embankment
32 gen table 1 ; mark top of ground surface
33 m null region 1,50 300,50 ; null region above marked ground surface - this will create
triangular elements along the face of the embankment
34 table 2 -162,0 -5,347 5,347 -64.8,0 -162,0 ;core
35 table 5 -54.8,0 10.98,328 22.2,328 153,0 -54.8,0; pit run gravel
36 gen table 2 ; mark core outline
37 gen line -172,0 -15.97,344.85 ; u/s edge of u/s gravel
38 gen table 5 ;mark pit run gravel outline
39 ;geometry adjustments
40 gen line 10.98,328 14.69,345.46
41 ini x -4.2931657 y 343.79724 i 199 j 110
42 ini x -5.0 y 347.0 i 198 j 111
43 ini x 5.0 y 347.0 i 203 j 111
44 ini x 14.865353 y 345.30142 i 208 j 110
45 ini x 10.966295 y 327.9948 i 205 j 106
46 ini x -16.360458 y 344.58615 i 193 j 110
47 ;assign groups to elements
48 group 'Gravel' ; assign all to gravel first and then reassign to groups that are better
defined
49 group 'Rockfill' region 143,68 231,68
50 group 'Core' region 189,65
51 group 'Pit Run' region 200,70
52 group 'Foundation' j = 1,15
53 group 'Gravel' i=183 j=16,30
54 group 'Gravel' i=184 j=37,49
55 group 'Gravel' i=185 j=60,63
56 group 'Gravel' i=194 j=16,24
57 group 'Gravel' i=195 j=30,47
58 group 'Gravel' i=196 j=58,62
59 group 'Gravel' i=197 j=110

```

```

60 ;Adjust for pool loading on nodes
61 ini x -34.170113 y 335.0 i 186 j 108
62 ini x 575.7984 y 24.6 i 265 j 22
63 save GEO.sav
64
65 ;Branch 2:STATIC.sav
66 ;-----;
67 ; Define Boundary Conditions and Initialize Stresses
68 ;-----;
69 fix x i=1 ; fix the edge of the model in the x dir
70 fix x,y j=1 ; fix the base of hte model in x and y directions
71 fix x i=$IGPMAX ; fix the other edge of the model in the x dir
72 history 9999 unbalanced ; assign history 9999 to track unbalanced force in model -
tracking equil
73 model elastic group 'Foundation' ; make foundation bedrock elastic
74 call groupPropsGrav.fis ; assign material properties
75 ; Define histories for tracking static analysis
76 his 1 srat ; track solve ratio
77 ;Solve for static stresses
78 ;Divide embankment into 10 equal increments by j zones
79 ;j 16 to 110 95 zones - 10 zones in each increment
80 ;1st lift j 16 26
81 model null i 1,$IGPMAX j 26,$JGPMAX
82 solve elastic
83 save STATIC.sav
84
85 ;Branch 3:LIFT_2.sav
86 ;-----
87 ;2nd lift j 26 36
88 ;-----
89 m mohr j 26,36
90 ini ydis = 0.0 xdis = 0.0 xvel = 0.0 yvel = 0.0
91 model null region 1,26 400,26
92 call groupPropsGrav.fis
93 solve
94 ;-----
95 ;3rd lift j 36 46
96 ;-----
97 m mohr j 36,46
98 ini ydis = 0.0 xdis = 0.0 xvel = 0.0 yvel = 0.0
99 model null region 1,36 400,36
100 call groupPropsGrav.fis
101 solve
102 ;-----
103 ;4th lift j 46 56
104 ;-----
105 m mohr j 46,56
106 ini ydis = 0.0 xdis = 0.0 xvel = 0.0 yvel = 0.0
107 model null region 1,46 400,46
108 call groupPropsGrav.fis
109 solve
110 ;-----
111 ;5th lift j 56 66
112 ;-----
113 m mohr j 56,66
114 ini ydis = 0.0 xdis = 0.0 xvel = 0.0 yvel = 0.0
115 model null region 1,56 400,56
116 call groupPropsGrav.fis
117 solve
118 ;-----
119 ;6th lift j 66 76
120 ;-----
121 m mohr j 66,76
122 ini ydis = 0.0 xdis = 0.0 xvel = 0.0 yvel = 0.0
123 model null region 1,66 400,66
124 call groupPropsGrav.fis
125 solve
126 ;-----
127 ;7th lift j 76 86

```

```

128 ;-----
129 m mohr j 76,86
130 ini ydis = 0.0 xdis = 0.0 xvel = 0.0 yvel = 0.0
131 model null region 1,76 400,76
132 call groupPropsGrav.fis
133 solve
134 ;-----
135 ;8th lift j 86 96
136 ;-----
137 m mohr j 86,96
138 ini ydis = 0.0 xdis = 0.0 xvel = 0.0 yvel = 0.0
139 model null region 1,86 400,86
140 call groupPropsGrav.fis
141 solve
142 ;-----
143 ;9th lift j 96 106
144 ;-----
145 m mohr j 96,106
146 ini ydis = 0.0 xdis = 0.0 xvel = 0.0 yvel = 0.0
147 model null region 1,96 400,96
148 call groupPropsGrav.fis
149 solve
150 ;-----
151 ;10th lift j 106 116
152 ;-----
153 m mohr j 106,116
154 ini ydis = 0.0 xdis = 0.0 xvel = 0.0 yvel = 0.0
155 model null region 1,106 400,106
156 model mohr i 191,192 j 109
157 call groupPropsGrav.fis
158 solve
159 save LIFT_2.sav
160
161 ;Branch 4:RES_1.sav
162 ;-----
163 ; Define permeabilities, this input is generated from permeabilites.xls
164 ;-----
165 prop k11 2.63E-8 k22 5.26E-8 group 'Core'
166 prop k11 5.26E-07 k22 5.26E-7 group 'Gravel'
167 prop k11 2.63E-05 k22 5.26E-06 group 'Pit Run'
168 prop k11 2.63E-05 k22 5.26E-06 group 'Rockfill'
169 prop k11 8.94E-08 k22 8.94E-08 group 'Foundation'
170 ; histories of u/s shell element
171 his 12 pp i=168 j=18
172 ; histories of core element
173 his 13 pp i=188 j=16
174 his 14 pp i=190 j=51
175 his 15 pp i=194 j=85
176 ;histoires of d/s pit run
177 his 16 pp i=205 j=16
178 his 17 pp i=205 j=51
179 his 18 pp i=205 j=85
180 water bulk = 4200 ; bulk modulus of water (note: can use reduced modulus to speed
flow).
181 Table 10 -1900,335 -34.2,335 -25.0,200.0 50.0,24.6 619.25,0 1900,24.6 ; assumed
phreatic surface
182 water table 10 ; apply water table
183 set funsat on
184 set flow on ; turn seepage flow on
185 set steps 10000000000
186 set sratio = 1e-6
187 cycle 1000 ;cycle model to smooth assumed phreatic
188 save RES_1.sav
189
190 ;Branch 5:MECH.sav
191 ;-----
192 ; Apply mechanical force of water
193 ;-----
194 ini xdis 0.0 ydis 0.0

```



```

195 ini xvel 0.0 yvel 0.0
196 ; Apply reservoir to elevation 1870 (335 FT) down to base of foundation El 1485 (-50)
197 ;d/s groundwater at ground surface el 1535 (0 ft) = tw
198 apply pres 24024 var 0,-24024 from 1,16 to 186,108 ; (1870-1485)*62.4 = 24024 psf
Reservoir Loading
199 set mech on ; turn mechanical on
200 set flow off ; turn seepage off
201 water bulk 0.0
202 set step 1000000
203 solve ; solve model
204 save MECH.sav
205
206 ;Branch 6:PRE_DY.sav
207 ;-----;
208 ; final dynamic properties
209 ;-----;
210 call groupPropsDyn.fis
211 groupPropsDyn ; update embankment materials for mean eff stress dependence
212 solve
213 step 1000
214 ini xdisp = 0.0 ydisp = 0.0
215 ;-----;
216 ; Prepare for Cycling (Bring to Equilibrium with Final Boundary Conditions)
217 ;-----;
218 set dyn=on
219 ; Initialize Damping
220 set dy_damp= combined
221 ; Define Itasca hysteretic damping
222 call hysdamping.dat ; call sig3 damping functions
223 ; Bring to Equilibrium with final fluid modulus
224 water bulk = 4100000 ; increase bulk modulus of water
225 ;call solvefmod5.dat ; REQUIRES FMOD5.FIS - NEED TO ADJUST UNITS FOR PSF?
226 ;-----;
227 ; Apply Free Field and Rigid Boundary Conditions Before Cycling
228 ;-----;
229 free x
230 fix j=1
231 apply ff
232 fix x y j=1
233 solve ; bring to equilibrium
234 step 1000
235 ;Initial values of x- and y-displacements/velocities
236 ini xdisp = 0.0 ydisp = 0.0
237 ini xvel = 0.0 yvel = 0.0
238 solve
239 ini xdisp = 0.0 ydisp = 0.0
240 ; Define Time Histories for dynamic analysis
241 his 2 dytime
242 his 10 xdis i=200 j=110
243 his 12 xdis i=200 j=16
244 his 11 ydis i=200 j=110
245 his 30 xacc i=200 j=110
246 his 31 xacc i=188 j=16
247 his 33 ydis i=189 j=16
248 his 34 ydis i=189 j=1
249 save PRE_DY.sav
250
251 ;Branch 7:PRE_CY.sav
252 ;-----;
253 ; State 5: Prepare for cycling - Read Earthquake Load
254 ;-----;
255 ; Prepare to cycle
256 ini xdisp = 0.0 ydisp = 0.0
257 ini xvel = 0.0 yvel = 0.0
258 set flow=on mech=on ; turn mech and flow on
259 set dytime=0.0 ; set dytime to 0 = start of record
260 set large
261 set step 1000000000
262 ; Set Damping

```

```

263 set dy_damp=rayleigh 0.01 1.0 ; apply a small amount of damping at the estimated
264 natural frequency to reduce numerical "noise"
265 ; Read and Convert Earthquake Record
266 set multi on
267 ; Filter acceleration time history
268 his read 6000 AKT023EW.acc; read rock outcrop acceleration time history
269 his write 6000 table 102 ; write in table
270 ; convert acc to m/sec2 (from "g") and divide by 2 (upward propagating motion)
271 call convert.fis
272 set tab_in 102 tab_out 103 npnts 3501 cfactor 12.558 ; 32.2/2 * 0.78 scaling factor
273 EDIT UNITS AS NEEDED AND SCALE FACTORS
274 convert
275 ;convert acceleration time history to velocity
276 call int.fis
277 set int_in 103 int_out 104
278 integrate
279 ca int.fis
280 ;Calculate shear stress time history for FLAC (i.e. 2 * shear stress)
281 def shearstress
282 dens = 141.0/32.2 ; SLUGS
283 Cs = 2493.0 ; half space Vs=2493 ft/s
284 fac = -2.0*dens*Cs ; A = -2.0?
285 command
286 set tab_in 104 tab_out 105 npnts 3501 cfactor fac
287 convert
288 endcommand
289 end
290 shearstress
291 ;Apply compliant base and shear stress time history at the base of the model
292 ; Apply compliant base
293 apply xquiet yquiet from 1,1 to $IGPMAX,1
294 ;Apply shear stress time history at the base of the model
295 apply sxxy 1.0 hist table 105 i=1,$IGPMAX j=1
296 save PRE_CY.sav
297
298 ;Branch 8:DY_FULLL.sav
299 Solve dytime = 35
300 save DY_FULLL.sav
301
302 ;Branch 9:dytime40.sav
303 solve dytime 40
304 save dytime40.sav
305
306 ;Branch 10:HIST.sav
307 set hisfile AKT023EW_ACC.his ; acceleration history at crest
308 his write 30 vs 2
309 set hisfile AKT023EWydsip.his ; ydisp at crest and foundation
310 his write 11 33 vs 2
311 set hisfile AKT023EWxdsip.his ; xdisp at crest and foundation
312 his write 3 10 vs 2
313 save HIST.sav

```

```
1 ;Name:convert
2 ;Diagram:
3 ;Input:tab_in/int/100/input table
4 ;Input:tab_out/int/101/output table
5 ;Input:npnts/int/4000/number of points in record
6 ;Input:cfactor/float/10.0/conversion factor from gs
7 def convert
8     loop ii (1,npnts)
9         ytable(tab_out,ii) = cfactor * ytable(tab_in,ii)
10        xtable(tab_out,ii) = xtable(tab_in,ii)
11    endloop
12 end
```

```

1  ; define properties for groups
2
3  ; there are no unit conversions for properties so all units should
4  ; be input to be consistent with input units:  ft, s, slugs
5
6  ;prop dens 3.820  shear=3.61e6  bulk=1.68e7  friction=13.5 cohesion=600  notnull group
   'Core'
7  ;prop dens 4.565  shear=1.32e7  bulk=2.85e7  friction=42.5  notnull group 'Gravel'
8  ;prop dens 4.379  shear=1.32e7  bulk=2.85e7  friction=42.5  notnull group 'Pit Run'
9  ;prop dens 4.752  shear=1.32e7  bulk=2.85e7  friction=45  notnull group 'Rockfill'
10 ;prop dens 4.379  shear=2.72e7  bulk=3.63e7  notnull group 'Foundation'
11
12 set echo off
13
14 ; function to define properties per group
15 def groupPropsDyn
16
17     gsolid = 2.7;
18     gw = 62.4 ; 64 pcf
19     g0 = 62.4 ; 62.4 pcf
20     pratl = 0.3 ; poisson's ratio
21
22     loop i (1,izones)
23         loop j (1,jzones)
24             if model(i,j) # 1
25                 _sm3_prime = -(syy(i,j)+sxx(i,j)+szz(i,j))/3.0 - pp(i,j) ; mean eff.
                    stress uses both calculated minor stress - plane strain
26                 _sm2_prime = -(syy(i,j)+sxx(i,j))/2.0 - pp(i,j) ; mean eff. stress uses
                    calculated only one minor stress dir
27
28                 if z_group(i,j) = 'Core' then ;
29                     uw = 123.0 ; total unit weight pcf
30                     cohesion(i,j) = 600 ;
31                     friction(i,j) = 13.5 ;
32                     su = -(esyy(i,j))*0.24 + cohesion(i,j)
33                     ;K2max = 80.0
34                     ;Gmax_psf = 1000.0 * K2max * sqrt(_sm3_prime*20.88) ;
35                     Gmax_psf = 600*su ; Gmax a function of Su
36                     vs0 = sqrt(Gmax_psf/(uw/32.2)) ; shear wave vel ft/s
37                     tension(i,j) = 0.0 ;
38                 endif
39                 if z_group(i,j) = 'Gravel' then ;
40                     uw = 147.0 ; total unit weight pcf
41                     K2max = 80.0
42                     Gmax_psf = 1000.0 * K2max * sqrt(_sm3_prime) ;
43                     vs0 = sqrt(Gmax_psf/(uw/32.2)) ; shear wave vel ft/s
44                     cohesion(i,j) = 0.0 ;
45                     friction(i,j) = 42.5 ;
46                     tension(i,j) = 0.0 ;
47                 endif
48                 if z_group(i,j) = 'Pit Run' then ;
49                     uw = 153.0 ; total unit weight pcf
50                     K2max = 80.0
51                     Gmax_psf = 1000.0 * K2max * sqrt(_sm3_prime) ;
52                     vs0 = sqrt(Gmax_psf/(uw/32.2)) ; shear wave vel ft/s
53                     cohesion(i,j) = 0.0 ;
54                     tension(i,j) = 0.0 ;
55                 endif
56                 if z_group(i,j) = 'Rockfill' then ;
57                     uw = 141.0 ; total unit weight pcf
58                     K2max = 80.0
59                     Gmax_psf = 1000.0 * K2max * sqrt(_sm3_prime) ;
60                     vs0 = sqrt(Gmax_psf/(uw/32.2)) ; shear wave vel ft/s
61                     cohesion(i,j) = 0 ;
62                     friction(i,j) = 45 ;
63                     tension(i,j) = 0.0 ;
64                 endif
65
66                 if z_group(i,j) = 'Foundation' then ;

```

```

67         vs0 = 2493.0 ; shear wave vel ft/s
68         uw = 141.0 ; total unit weight pcf
69         cohesion(i,j) = 5000 ;
70         friction(i,j) = 50. ;
71     endif
72
73     ; calculate density, moduli, porosity
74     vratio= (uw/g0-gsolid)/(gw/g0-uw/g0);
75     porosity(i,j)=vratio/(1+vratio);
76     dryden=gsolid*g0/(1+vratio);
77     density(i,j)=dryden/32.2;
78     dum = uw/32.2;
79     shear_mod(i,j) = vs0^2 * dum;
80     bulk_mod(i,j) = shear_mod(i,j)*(2.*(1.+prat))/(3.*(1.-2.*prat));
81     end_if
82     end_loop
83 end_loop
84 end
85
86 groupPropsDyn
87
88 set echo on
89

```

```

1 ; define properties for groups
2
3
4 ; input units: ft, s, slugs
5 ; parameters needed: dry density, shear modulus, bulk modulus, friction angle, cohesion
6 ; Core:      sat UW = 123 pcf, dry UW = 95 pcf, Gmax = 600Su, Poisson's ratio =
0.4, Su = 600 psf, phi = 13.5
7 ; Gravel:    sat UW = 147 pcf, dry UW = 132 pcf, k2max = 120, Poisson's ratio =
0.3, phi = 42.5 deg
8 ; Pit Run:   sat UW = 153 pcf, dry UW = 142.5 pcf, k2max = 120, Poisson's ratio = 0.3,
phi = 42.5
9 ; Rockfill:  sat UW = 141 pcf, dry UW = 127 pcf, k2max = 120, Poisson's ratio = 0.3,
phi = 45
10 ; Bedrock:   sat UW = 141 pcf, dry UW = 127 pcf, Vs = 2493 fps, Poisson's ratio =
0.2, elastic model
11 ;prop dens 3.820 shear=3.61e6 bulk=1.68e7 friction=13.5 cohesion=600 notnull group
'Core'
12 ;prop dens 4.565 shear=1.32e7 bulk=2.85e7 friction=42.5 notnull group 'Gravel'
13 ;prop dens 4.379 shear=1.32e7 bulk=2.85e7 friction=42.5 notnull group 'Pit Run'
14 ;prop dens 4.752 shear=1.32e7 bulk=2.85e7 friction=45 notnull group 'Rockfill'
15 ;prop dens 4.379 shear=2.72e7 bulk=3.63e7 notnull group 'Foundation'
16
17 ; function to define properties per group
18 def groupPropsGrav
19
20     gsolid = 2.7;
21     gw = 62.4 ; kN/m3 (64 pcf)
22     g0 = 62.4 ; kN/m3 (62.4 pcf)
23     prat1 = 0.3 ; poisson's ratio
24
25
26     loop i (1,izones)
27         loop j (1,jzones)
28             if model(i,j) # 1
29
30                 if z_group(i,j) = 'Core' then ;
31                     ; for now use constant value, then make pressure dependent
32                     vs0 = 972.0 ; shear wave vel ft/s
33                     uw = 123.0 ; total unit weight pcf
34                     cohesion(i,j) = 600 ;
35                     friction(i,j) = 13.5 ;
36                 endif
37                 if z_group(i,j) = 'Gravel' then ;
38                     ; for now use constant value, then make pressure dependent
39                     vs0 = 1295.0 ; shear wave vel ft/s
40                     uw = 147.0 ; total unit weight pcf
41                     cohesion(i,j) = 0 ;
42                     friction(i,j) = 42.5 ;
43                 endif
44                 if z_group(i,j) = 'Pit Run' then ;
45                     ; for now use constant value, then make pressure dependent
46                     vs0 = 1270.0 ; shear wave vel ft/s
47                     uw = 153.0 ; total unit weight pcf
48                     cohesion(i,j) = 0 ;
49                     friction(i,j) = 42.5 ;
50                 endif
51                 if z_group(i,j) = 'Rockfill' then ;
52                     ; for now use constant value, then make pressure dependent
53                     vs0 = 1143.0 ; shear wave vel ft/s
54                     uw = 141.0 ; total unit weight pcf
55                     cohesion(i,j) = 0 ;
56                     friction(i,j) = 45.0 ;
57                 endif
58                 if z_group(i,j) = 'Foundation' then ;
59                     ; for now use constant value, then make pressure dependent
60                     vs0 = 2493.0 ; shear wave vel ft/s
61                     uw = 141.0 ; total unit weight pcf
62                     cohesion(i,j) = 0 ;
63                     friction(i,j) = 45. ;

```

```

64         endif
65         ; calculate density, moduli, porosity
66         vratio= (uw/g0-gsolid)/(gw/g0-uw/g0);
67         porosity(i,j)=vratio/(1+vratio);
68         dryden=gsolid*g0/(1+vratio);
69         density(i,j)=dryden/32.2;
70         dum = uw/32.2;
71         ; cohesion(i,j) = 5.0e10 ;
72         ; tension(i,j) = 5.0e10 ;
73         shear_mod(i,j) = vs0^2 * dum;
74         bulk_mod(i,j) = shear_mod(i,j)*(2.*(1.+prat))/(3.*(1.-2.*prat));
75     end_if
76 end_loop
77 end_loop
78 end
79
80 groupPropsGrav
81
82 set echo on
83

```

```
1 ;hysdamping.dat
2 ini dy_damp hyst sig3 1.02 -0.60 -1.80 group 'Rockfill' ; from fitting Gazetas (1992)
  rockfill curve
3 ini dy_damp hyst sig3 1.014 -0.4792 -1.15 group 'Pit Run' ; from fitting EPRI 50-120 ft
4 ini dy_damp hyst sig3 1.014 -0.4792 -1.15 group 'Gravel' ; from fitting EPRI 50-120 ft
5 ini dy_damp hyst sig3 1.014 -0.52 -1.22 group 'Gravel' ; from fitting Vucetic-Dobry
  (1991) PI = 15
6
7
8
9
```



```

1  ;Code: FLAC
2  ;Name: integrate
3  ;Purpose: Integrate a table
4  ;Diagram:
5  ;Input: int_in/int/1/table with original data
6  ;Input: int_out/int/2/table with integration
7  ;Note: This routine will integrate a table, outputting another table
8  ;Note: containing the integration. The resulting table will have the
9  ;Note: same number of points as the original.
10 ;
11 def integrate
12     command
13         table int_out erase
14     end_command
15     nitem = table_size(int_in)
16 ;
17     xold = xtable(int_in,1)
18     yold = ytable(int_in,1)
19     val = 0.0
20     xtable(int_out,1) = xold
21     ytable(int_out,1) = val
22     loop ii (2,nitem)
23         xnew = xtable(int_in,ii)
24         ynew = ytable(int_in,ii)
25         val = val + 0.5*(yold + ynew)*(xnew-xold)
26         xtable(int_out,ii) = xnew
27         ytable(int_out,ii) = val
28         xold = xnew
29         yold = ynew
30     end_loop
31 ;
32 end
33 ;
34 SUB

```